Evaluating Scour at Bridges

HEC 18, Third Edition
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Metric Version

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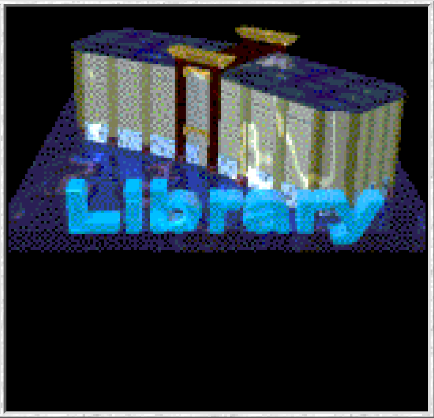

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Chapter 2 : HEC 18

Basic Concepts and Definitions of Scour

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2.1 General

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour-resistant. **However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams.** Under constant flow conditions, scour will reach maximum depth in sand- and gravel-bed material in hours; cohesive bed material in days; glacial till, sandstones, and shale in months; limestone in years, and dense granite in centuries. Under flow conditions typical of actual bridge crossings, several floods may be needed to attain maximum scour.

Determining the magnitude of scour is complicated by the cyclic nature of the scour process. Scour can be deepest near the peak of a flood, but hardly visible as floodwaters recede and scour holes refill with sediment.

Designers and inspectors need to carefully study site-specific subsurface information in evaluating scour potential at bridges, giving particular attention to foundations on rock. Massive rock formations with few discontinuities are highly resistant to scour during the lifetime of a typical bridge.

All of the equations for estimating contraction and local scour are based on laboratory experiments with limited field verification. However, contraction and local scour depths at piers as deep as computed by these equations have been observed in the field. The equations recommended in this document are considered to be the most applicable for estimating scour depths.

A factor in scour at highway crossings and encroachments is whether it is clear-water or live-bed scour. Clear-water scour occurs where there is no transport of bed material upstream of the crossing or encroachment or the material being transported in the upstream reach is transported through the downstream reach at less than the capacity of the flow. Live-bed scour occurs where there is transport of bed material from the upstream reach into the crossing or encroachment. This subject is discussed in Section 2.4 and Section 2.6.

This document presents procedures, equations, and methods to analyze scour in both riverine and coastal areas. In riverine environments, scour results from flow in one direction (downstream). In coastal areas, highways that cross streams and/or encroach longitudinally on them are subject to tidal fluctuation and scour may result from flow in two directions. In waterways influenced by tidal fluctuations, flow velocities do not necessarily decrease as scour occurs and the waterway area increases. In tidal waterways as waterway area increases, the discharge may increase. This is in sharp contrast to riverine waterways where the principle of flow continuity and a constant discharge requires that velocity be inversely proportional to the waterway area. **However, the methods and equations for determining stream instability, scour and associated countermeasures can be applied to both riverine and coastal environments.**[10,11] The difficulty in tidal streams is in determining the hydraulic parameters (such as discharge, velocity, and depth) that are to be used in the scour equations. Tidal scour is discussed in Chapter 4.

2.2 Total Scour

Total scour at a highway crossing is comprised of three components:

1. Long-term aggradation or degradation,
2. Contraction scour, and
3. Local scour.

In addition, lateral migration of the channel must be assessed when evaluating total scour at bridge piers and abutments.

2.2.1 Aggradation and Degradation

Aggradation and degradation are long-term streambed elevation changes due to natural or man-induced causes which can affect the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge; whereas, degradation involves the lowering or scouring of the streambed due to a deficit in sediment supply from upstream.

2.2.2 Contraction Scour

Contraction scour, in a natural channel or at a bridge crossing, involves the removal of material from the bed and banks across all or most of the channel width. This component of scour can result from a contraction of the flow area, an increase in discharge at the bridge, or both. It can also result from a change in downstream control of the water surface elevation. The scour is the result of increased velocities and shear stress on the channel bed.

Contraction of the flow by bridge approach embankments encroaching onto the floodplain and/or into the main channel is the most common cause of contraction scour. Contraction scour can be either clear-water or live-bed. Live-bed contraction scour occurs when there is transport of bed material in the approach reach; whereas, clear-water contraction scour occurs when there is no bed material transport in the approach reach or the bed material being transported in the upstream reach is so fine that it washes through the contracted section. Live-bed contraction scour typically occurs during the rising stage of a runoff event, while refilling of the scour hole occurs during the falling stage. Also, clear-water scour at low or moderate flows can change to live-bed scour at high flows. This cyclic nature creates difficulties in measuring contraction scour after a flood event.

2.2.3 Local Scour

Local scour involves removal of material from around piers, abutments, spurs, and embankments. It is caused by an acceleration of flow and resulting vortices induced by the flow obstructions. Local scour can also be either clear-water or live-bed scour. Live-bed local scour is cyclic in nature; that is, the scour hole that develops during the rising stage refills during the falling stage.

2.2.4 Lateral Stream Migration

In addition to the types of scour mentioned above, naturally occurring lateral migration of the main channel of a stream within a floodplain may increase pier scour, erode abutments or the approach roadway, or change the total scour by changing the flow angle of attack at piers. Factors that affect lateral stream movement also affect the stability of a bridge. These factors are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials (see Hydraulic Engineering Circular No. 20, and "Highways in the River Environment").[12,13]

The following sections provide a more detailed discussion of the various components of total scour.

2.3 Aggradation and Degradation - Long-Term Streambed Elevation Changes

Long-term bed elevation changes may be the natural trend of the stream or may be the result of some modification to the stream or watershed. The streambed may be aggrading, degrading, or in relative equilibrium in the vicinity of the bridge crossing. In this section, long-term trends are considered. Long-term aggradation and degradation do not include the localized cutting and filling of the streambed that might occur during a runoff event (contraction and local scour). A stream may cut and fill at specific locations during a runoff event and also have a long-term trend of an increase or decrease in bed elevation over a longer reach of a stream. The problem for the engineer is to estimate the long-term bed elevation changes that will occur during the life of the structure.

A long-term trend may change during the life of the bridge. These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or human activities. The engineer must assess the present state of the stream and watershed and then evaluate potential future changes in the river system. From this assessment, the long-term streambed changes must be estimated.

Factors that affect long-term bed elevation changes are dams and reservoirs (up- or downstream of the bridge), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoffs of meander bends (natural or man-made), changes in the downstream channel base level (control), gravel mining from the streambed, diversion of water into or out of the stream, natural lowering of the fluvial system, movement of a bend, bridge location with respect to stream planform, and stream movement in relation to the crossing. Tidal ebb and flood may degrade a coastal stream; whereas, littoral drift may result in aggradation.

The U.S. Army Corps of Engineers (USACOE), the U.S. Geological Survey (USGS) and other Federal and State agencies should be contacted concerning documented long-term streambed variations. If no data exist or if such data require further evaluation, an assessment of long-term streambed elevation changes for riverine streams should be made using the principles of river mechanics. Such an assessment requires the consideration of all influences upon the bridge crossing, i.e., runoff from the watershed to a stream (hydrology), sediment delivery to the channel (watershed erosion), sediment transport capacity of a stream (hydraulics), and response of a stream to these factors (geomorphology and river mechanics). With coastal streams, the principles of both river and coastal engineering mechanics are needed. In coastal streams, estuaries or inlets, in addition to the above, consideration must be given to tidal conditions, i.e., the magnitude and period of the storm surge, sediment delivery to the channel by the ebb and flow of the tide, littoral drift, sediment transport capacity of the tidal flows, and response of the stream, estuary, or inlet to these tidal and coastal engineering factors.

Significant morphologic impacts can result from human activities. The assessment of the impact of human activities requires a study of the history of the river, estuary, or tidal inlet, as well as a study of present water and land use and stream control activities. All agencies involved with the river or coastal area should be contacted to determine possible future changes.

To organize such an assessment, a three-level fluvial system approach can be used consisting of:

1. a qualitative determination based on general geomorphic and river mechanics relationships,
2. an engineering geomorphic analysis using established qualitative and quantitative relationships to estimate the probable behavior of the stream system to various scenarios or future conditions, and
3. physical models or physical process computer modeling using mathematical models such as BRI-STARS and the USACOE HEC-6 to make predictions of quantitative changes in streambed elevation due to changes in the stream and watershed.(14,15) Methods to be used in Levels 1 and 2 are presented in HEC-20, "Stream Stability at Highway Structures," and HIRE.(12,13) Additional discussion of this subject is presented in Chapter 4.

For coastal areas, where highway crossings (bridges) and/or longitudinal stream encroachments are subject to tidal influences, the three-level approach used in fluvial systems is also appropriate. The approach for tidal waterways is described in Chapter 4 of this document.

2.4 Contraction Scour

2.4.1 General

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or bridge. It also occurs when overbank flow is forced back to the channel by roadway embankments at the approaches to a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until relative equilibrium is reached; i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach, or the bed shear stress is decreased to a value such that no sediment is transported out of the reach.

In coastal waterways which are affected by tides, as the cross-sectional area increases the discharge from the ocean may increase and thus the velocity and shear stress may not decrease. Consequently, relative equilibrium may not be reached. Thus, at tidal inlets contraction scour may result in a continual lowering of the bed (long-term degradation).

Contraction scour can also be caused by short-term (daily, weekly, yearly, or seasonal) changes in the downstream water surface elevation that control backwater and hence, the velocity through the bridge opening. Because this scour is reversible, it is considered contraction scour rather than long-term aggradation or degradation.

Contraction scour is typically cyclic; for example, the bed scours during the rising stage of a runoff event and fills on the falling stage. The contraction of flow due to a bridge can be caused by either a natural decrease in flow area of the stream channel or by abutments projecting into the channel and/or piers blocking a portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off floodplain flow. This can cause clear-water scour on a setback portion of a bridge section or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments. This clear-water picks up additional sediment from the bed upon reaching the bridge opening. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow returning to the main channel at the end of the abutment. The difference between clear-water and live-bed scour is discussed further in Section 2.7.

Other factors that can cause contraction scour are

1. natural stream constrictions,
2. long highway approaches to the bridge over the floodplain,
3. ice formations or jams,
4. natural berms along the banks due to sediment deposits,
5. islands or bar formations up- or downstream of the bridge opening,
6. debris, and

7. vegetative growth in the channel or floodplain.

In a natural channel, the depth of flow is always greater on the outside of a bend. In fact, there may well be deposition on the inner portion of the bend at a point bar. If a bridge is located on or close to a bend, the contraction scour will be concentrated on the outer portion of the bend. Also, in bends, the thalweg (the part of the stream where the flow is deepest and, typically, the velocity is the greatest) may shift toward the center of the stream as the flow increases. This can increase scour and the nonuniform distribution of scour in the bridge opening.

Contraction Scour Equations. There are two forms of contraction scour depending upon the competence of the uncontracted approach flow to transport bed material into the contraction. Live-bed scour occurs when there is streambed sediment being transported into the contracted section from upstream. In this case, the scour hole reaches equilibrium when the transport of bed material out of the scour hole is equal to that transported into the scour hole from upstream. Clear-water scour occurs when the bed material sediment transport in the uncontracted approach flow is negligible or the material being transported in the upstream reach is transported through the downstream reach at less than the capacity of the flow. In this case, the scour hole reaches equilibrium when the average bed shear stress is less than that required for incipient motion of the bed material. Clear-water and live-bed scour are discussed further in Section 2.7.

Contraction scour equations are based on the principle of conservation of sediment transport (continuity). In the case of live-bed scour, the fully developed scour in the bridge cross section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For live-bed scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For clear-water scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section.

2.4.2 Live-Bed Contraction Scour Equation

Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in. Normally, the width of the contracted section is constrained and depth increases until the limiting conditions are reached.

Laursen derived the following live-bed contraction scour equation based on a simplified transport function, transport of sediment in a long contraction, and other simplifying assumptions.[16] The application of this equation is presented in Section 4.3.4.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \left(\frac{n_2}{n_1}\right)^{k_2} \qquad (1)$$

$$y_3 = y_2 - y_o = \text{(Average scour depth, m)} \qquad (2)$$

where:

y_1 = Average depth in the upstream main channel, m
y_2 = Average depth in the contracted section, m
y_o = Existing depth in the contracted section before scour, m
Q_1 = Flow in the upstream channel transporting sediment, m^3/s
Q_2 = Flow in the contracted channel, m^3/s. Often this is equal to the total discharge unless the total flood flow is reduced by relief bridges, water overtopping the approach roadway, or in the setback area
W_1 = Bottom width of the upstream main channel, m
W_2 = Bottom width of main channel in the contracted section, m
n_2 = Manning's n for contracted section
n_1 = Manning's n for upstream main channel
k_1 & k_2 = Exponents determined below depending on the mode of bed material transport
V_* = $(gyS_1)^{1/2}$ shear velocity in the upstream section, m/s
ω = Median fall velocity of the bed material based on the D_{50}, m/s (see Figure 3 in Chapter 4)
g = Acceleration of gravity (9.81 m/s^2)
S_1 = Slope of energy grade line of main channel, m/m
D_{50} = Median diameter of the bed material, m

V_*/w	k_1	k_2	Mode of Bed Material Transport
<0.50	0.59	0.066	Mostly contact bed material discharge
0.50 to 2.0	0.64	0.21	Some suspended bed material discharge
>2.0	0.69	0.37	Mostly suspended bed material discharge

The location of the upstream section for y_1, Q_1, W_1, and n_1 needs to be located with engineering judgment. If WSPRO is used to obtain the values of the quantities, then the upstream channel section is located a distance equal to one bridge opening from the upstream face of the bridge.

2.4.3 Clear-Water Contraction Scour Equations

Clear-water contraction scour occurs in a long contraction when (1) there is no bed material transport from the upstream reach into the downstream reach or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of the flow. With clear-water contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (τ_o) on the bed is equal to the critical velocity (V_c) or the critical shear stress (τ_c) of a certain particle size (D) in the bed material. Normally, the width (W) of the contracted section is constrained and the depth (y) increases until the limiting conditions are reached.

Following a development given by Laursen (1963) equations for determining the clear-water contraction scour in a long contraction were developed in metric units.(17) For equilibrium in the contracted reach:

$$\tau_o = \tau_c \qquad (3)$$

where:

τ_o = Average bed shear stress, contracted section, Pa (N/m^2)
τ_c = Critical bed shear stress at incipient motion, Pa (N/m^2)

The average bed shear stress using y for the hydraulic radius (R) and Manning's equation to determine the slope (S_f) can be expressed as follows:

$$\tau_o = \gamma \; y \; S_f = \frac{\rho \; g \; n^2 \; V^2}{y^{1/3}} \qquad (4)$$

For noncohesive bed materials and fully developed clear-water contraction scour, the critical shear stress can be determined using Shields relation(13,17),

$$t_c = K_s (r_s - r) g D \qquad (5)$$

The bed in a long contraction scours until $\tau_o = \tau_c$ resulting in:

$$\frac{\rho \; g \; n^2 V^2}{y^{1/3}} = k_s (\rho_s - \rho) g \; D \qquad (6)$$

Solving for the depth (y) in the contracted section gives

$$y = \left[\frac{n^2\ V^2}{K_s(S_s - 1)D}\right]^3 \qquad (7)$$

In terms of discharge (Q) the depth (y) is:

$$y = \left[\frac{n^2 Q^2}{K_s(S_s - 1)D\ W^2}\right]^{3/7} \qquad (8)$$

where:

y =	Average depth in the contracted section after contraction scour, m
S_f =	Slope of the energy grade line, m/m
V =	Average velocity in the contracted section, m/s
D =	Diameter of smallest nontransportable particle in the bed material, m
Q =	Discharge, m^3/s
W =	Bottom width of contracted section, m
g =	Acceleration of gravity (9.81 m/s^2)
n =	Manning's roughness coefficient
K_s =	Shield's coefficient
S_s =	Specific gravity (2.65 for quartz)
γ =	Unit weight of water (9800 N/m^3)
ρ =	Density of water (1000 kg/m^3)
ρ_s =	Density of sediment (quartz, 2647 kg/m^3)

Equation 7 and Equation 8 are the basic equations for the clear-water scour depth (y) in a long contraction. Laursen, in English units used a value of 4 for K_s $(\rho_s$-$\rho)g$ in Equation 5; D_{50} for the size (D) of the smallest nonmoving particle in the bed material and Strickler's approximation for Manning's n ($n = 0.034\ D_{50}^{1/6}$).[(17)] Laursen's assumption that $\tau_c = 4\ D_{50}$ with $S_s = 2.65$ is equivalent to assuming a Shield's parameter $K_s = 0.039$.

From experiments in flumes and studies in natural rivers with bed material of sand, gravel cobbles, and boulders, Shield's coefficient (K_s) to initiate motion ranges from 0.01 to 0.25 and is a function of particle size, Froude

Number, and size distribution.[18,19,20,21,22,23] Some typical values for K_s for Fr. < 0.8 and as a function of bed material size are (1) K_s = 0.047 for sand (D_{50} from 0.065 to 2.0 mm); (2) K_s = 0.03 for median coarse-bed material (2 mm > D_{50} < 40 mm) and (3) K_s = 0.02 for coarse-bed material (D_{50} > 40 mm).

In metric units, Strickler's equation for n as given by Laursen is 0.041 $D_{50}^{1/6}$, where D_{50} is in meters. Research discussed in HIRE recommends the use of the effective mean bed material size (D_m) in place of the D_{50} size for the beginning of motion (D_{50} = 1.25 D_m). Changing D_{50} to D_m in the Strickler's equation gives n = 0.040 $D_m^{1/6}$.[13] Substituting K_s = 0.039 into Equation 7 and Equation 8 gives the following equations for y:

$$y = \left[\frac{V^2}{40\ D_m^{2/3}}\right]^3 \qquad (9)$$

$$y = \left[\frac{Q^2}{40\ D_m^{2/3}\ W^2}\right]^{3/7} \qquad (10)$$

$$y_s = y - y_o = (\text{average scour depth, m}) \qquad (11)$$

where:

Q =Discharge through contraction, m^3/s
D_m =Diameter of the bed material (1.25 D_{50}) in the contracted section, m
W =Bottom width in contraction, m
y_o =Existing depth in the contracted section before scour, m

The clear-water contraction scour equations assume homogeneous bed materials. However, with clear-water scour in stratified materials, using the layer with the finest D_{50} would result in the most conservative estimate of contraction scour. Alternatively, the clear-water contraction scour equations could be used sequentially for stratified bed materials. An example problem illustrating the use of the contraction scour equation is presented in Chapter 4.

Equation 8 and Equation 10 do not give the distribution of the contraction scour in the cross section. In many cases, assuming a uniform contraction scour depth across the opening would not be in error (e.g., short bridges, relief bridges and bridges, with simple cross sections and on straight reaches.) However, for wide bridges, bridges on bends, bridges with large overbank flow, or crossings with a large variation in bed material size distribution, the contraction scour depths will not be uniformly distributed across the bridge opening. In these cases, Equation 7 or Equation 9 can be used if the distribution of the velocity and/or the bed material is known. The computer program WSPRO uses stream tubes to give the discharge and velocity distribution in the cross section.(24) Using this distribution, Equation 7 or Equation 9 can be used to estimate the distribution of the contraction scour depths. Equation 8 and Equation 10 are used to determine the average contraction scour depth in the section.

Both the live-bed and clear-water contraction scour equations are the best that are available and should be regarded as a first level of analysis. If a more detailed analysis is warranted, a sediment transport model like BRI-STARS could be used.(14)

2.4.4 Critical Velocity of the Bed Material

The velocity and depth given in Equation 7 are associated with initiation of motion of the indicated particle size (D). Rearranging Equation 7 to give the critical velocity (V_c) for beginning of motion of bed material of size D results in:

$$V_c = \frac{K_s^{1/2}(S_s - 1)^{1/2}\ D^{1/2}\ y^{1/6}}{n} \tag{12}$$

Using $K_s = 0.039$, $S_s = 2.65$, and $n = 0.041\ D^{1/6}$

$$V_c = 6.19\ y^{1/6}\ D^{1/3} \tag{13}$$

where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, m/s
K_s = Shields parameter
S_s = Specific gravity of the bed material
D = Size of bed material, m
y = Depth of flow, m
n = Manning's roughness coefficient

2.5 Backwater

The **live-bed** contraction scour equation is derived assuming a uniform reach above and a long contraction into a uniform reach below the bridge. With **live-bed scour** the contraction scour equation computes a depth after the long contraction where the sediment transport into the downstream reach is equal to the sediment transport out of the downstream reach. The **clear-water** contraction scour equations are derived assuming that the depth at the bridge increases until the shear stress and velocity are decreased so that there is no longer any sediment transport. It is further assumed that flow goes from one uniform flow condition to another. Because of the assumption of a long contraction, the equations can over-estimate the scour depth. However, there are accelerations of the flow at the bridge that will offset the error introduced by the assumption of going from one uniform flow condition to another. In addition, if there is appreciable backwater above the bridge, the flow acceleration through the bridge will increase the scour at the bridge, tending to counteract the fact that the equations compute scour depths for uniform flow conditions.

Laursen did not use an energy balance in the derivation of the live-bed contraction scour equation.[25] However, one could draw a control volume with a backwater increment and still derive Laursen's relationship for y_2/y_1 using his assumptions. Also, computing $y_s = y_2 - y_1$, assumes a level water surface and a more consistent computation would be to write an energy balance before and after the scour. For **live-bed** conditions, the energy balance would be between the approach section (1) and the contracted section (2). Whereas, for **clear-water** scour, it would be the energy at the same section before (1) and after (2) the contraction scour. The energy balance yields the following equation:

$$y_s = (y_2 - y_1) + (V_2^2/2g - V_1^2/2g) - S_o L_{1-2} + h_{1-2} \qquad (14)$$

where:

S_o = the average bed slope (m/m)
h_{1-2} = the head loss from sections 1 and 2 (m).

The contraction scour equations simply ignore the last three terms which are normally very small compared to the contraction scour. **However, if the contraction causes ponding upstream and the flow at the contraction has a high velocity, the last three terms may not be small, especially the velocity head term.** At the limit, the velocity may be so large that the Froude Number in the contracted section equals 1.0 and an undular jump forms downstream of the bridge. **Engineering judgment needs to be used when analyzing backwater conditions.**

Backwater, in extreme cases, can decrease the velocity, shear stress and the sediment transport in the upstream section. This will increase the scour at the contracted section. The backwater can, by storing sediment in the upstream section, change **live-bed** scour to **clear-water** scour.

The contraction scour equations give the first cut, worst-case, simplified computations. However, the equations calculate contraction scour depths that have been observed in the field.

2.6 Local Scour

The basic mechanism causing local scour at piers or abutments is the formation of vortices (known as the horseshoe vortex) at their base (Figure 1). The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or abutment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is reestablished between bed material inflow and outflow and scouring ceases. For clear-water scour, scouring ceases when the shear stress caused by the horseshoe vortex equals the critical shear stress of the sediment particles at the bottom of the scour hole.

In addition to the horseshoe vortex around the base of a pier, there are vertical vortices downstream of the pier called the wake vortex (Figure 1). Both the horseshoe and wake vortices remove material from the pier base region. However, the intensity of wake vortices diminishes rapidly as the distance downstream of the pier increases. Therefore, immediately downstream of a long pier there is often deposition of material.

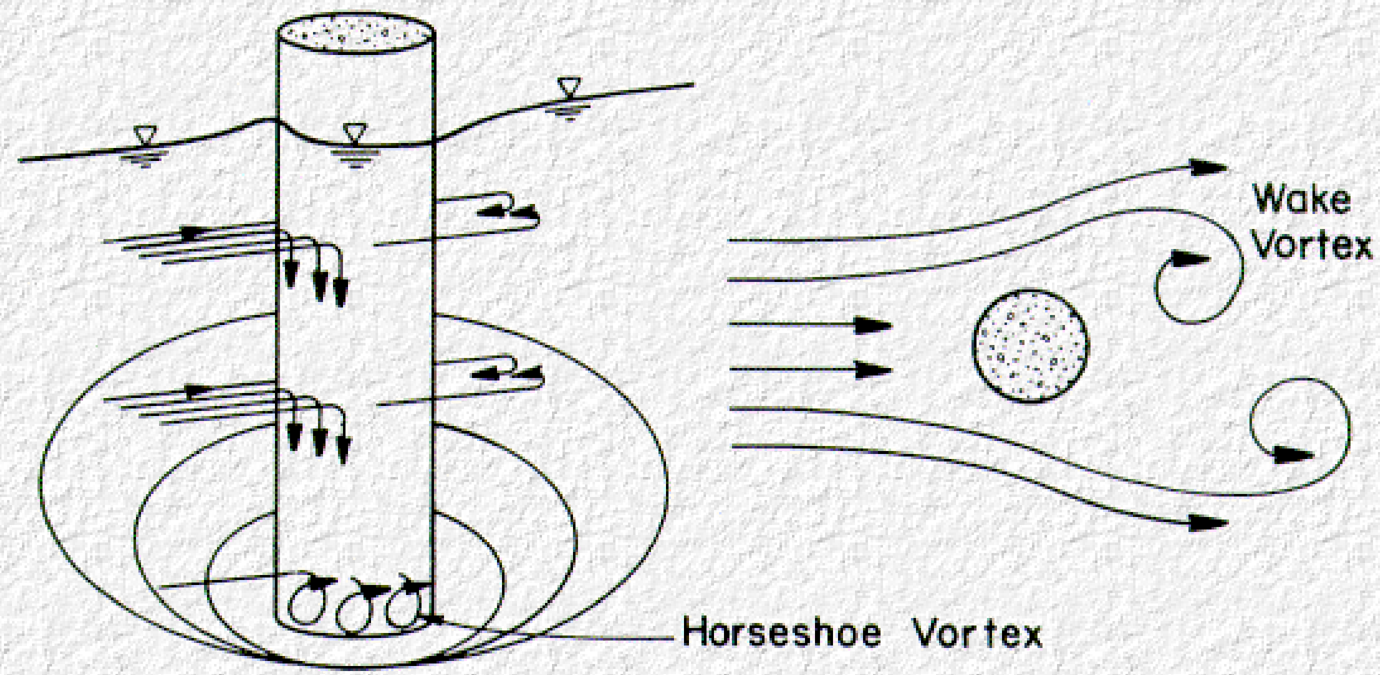

Figure 1. Schematic Representation of Scour at a Cylindrical Pier

Factors which affect the magnitude of local scour depth at piers and abutments are

1. velocity of the approach flow,
2. depth of flow,
3. width of the pier,
4. discharge intercepted by the abutment and returned to the main channel at the abutment (in laboratory flumes this discharge is a function of projected length of an abutment into the flow),
5. length of the pier if skewed to flow,
6. size and gradation of bed material,
7. angle of attack of the approach flow to a pier or abutment,
8. shape of a pier or abutment,
9. bed configuration, and
10. ice formation or jams and debris.

1. Flow velocity affects local scour depth. The greater the velocity, the deeper the scour. There is a high probability that scour is affected by whether the flow is subcritical or supercritical. However, most research and data are for subcritical flow (i.e., flow with a Froude Number less than 1.0, $Fr < 1$).

2. Flow depth also has an influence on the depth of local scour. An increase in flow depth can increase scour depth by a factor of 2 or greater for piers. With abutments, the increase is approximately 1.1 to 2.15 depending on the shape of the abutment.

3. Pier width has a direct influence on depth of local scour. As pier width increases, there is an increase in scour depth. There is a limit to the increase in scour depth as width increases. Very wide piers (piers wider than 10 m) do not have scour depths as deep as predicted by existing equations.

4. In laboratory flume studies, an increase in the projected length of an abutment (or embankment) into the flow increased scour; whereas, this is not the case in the field. This result for flumes is caused by the fact that the discharge intercepted by the abutment and returned to the main channel is a function of the abutment length. However, in the field case with a nonuniform distribution of flow, the discharge returned to the main channel is not simply a function of the abutment length. Because of this, abutment scour equations, which are based on laboratory experiments, give very large depths. These depths would occur in the field only for conditions that duplicate the conditions under which the flume experiments were conducted.

5. Pier length has no appreciable effect on local scour depth as long as the pier is aligned with the flow. When the pier is skewed to the flow, the pier length has a significant influence on scour depth. For example, doubling the length of the pier increases scour depth from 30 to 60 percent (depending on the angle of attack).

6. Bed material characteristics such as size, gradation, and cohesion can affect local scour. Bed material in the sand-size

range has little effect on local scour depth. Likewise, larger size bed material that can be moved by the flow or by the vortices and turbulence created by the pier or abutment will not affect the maximum scour, but only the time it takes to attain it. Very large particles in the bed material, such as coarse gravels, cobbles or boulders, may armor the scour hole. Research at the University of Aukland, New Zealand, by the Washington State Department of Transportation, and by other researchers developed equations that take into account the decrease in scour due to the armoring of the scour hole.[26,27,28,29] Richardson and Richardson combined these equations into a simplified equation, which accounted for bed material size.[30] However, field data are inadequate to support these equations at this time.

Molinas in flume experiments sponsored by FHWA, showed for Froude Numbers less than 1.0 ($Fr < 1.0$), and a range of bed material sizes, that when the approach velocity (V_1) of the flow is less than the critical velocity (V_c) of the D_{90} size of the bed material, the D_{90} size will decrease the scour depth.[31] Richardson and Richardson proposed a correction coefficient K_4 to the equation given in HEC 18 based on Molinas' results and Jones developed an equation for K_4.[25,32]

The size of the bed material also determines whether the scour at a pier or abutment is clear-water or live-bed scour. This topic is discussed in Section 2.7.

Fine bed material (silts and clays) will have scour depths as deep as sand-bed streams. This is true even if bonded together by cohesion. The effect of cohesion is to influence the time it takes to reach maximum scour. With sand-bed material the time to reach maximum depth of scour is measured in hours and can result from a single flood event. With cohesive bed materials it may take much longer to reach the maximum scour depth, the result of many flood events.

7. Angle of attack of the flow to the pier or abutment has a significant effect on local scour, as was pointed out in the discussion of pier length. Abutment scour is reduced when embankments are angled downstream and increased when embankments are angled upstream. According to the work of Ahmad, the maximum depth of scour at an embankment inclined 45 degrees downstream is reduced by 20 percent; whereas, the maximum scour at an embankment inclined 45 degrees upstream is increased about 10 percent.[33]

8. Shape of the nose of a pier or an abutment can have up to a 20 percent influence on scour depth. Streamlining the front end of a pier reduces the strength of the horseshoe vortex, thereby reducing scour depth. Streamlining the downstream end of piers reduces the strength of the wake vortices. A square-nose pier will have maximum scour depths about 20 percent greater than a sharp-nose pier and 10 percent greater than either a cylindrical or round-nose pier. The shape effect is negligible for flow angles in excess of five degrees. Full retaining abutments with vertical walls on the stream side (parallel to the flow) and vertical walls parallel to the roadway will produce scour depths about double that of spill-through (sloping) abutments.

9. Bed configuration of sand-bed channels affects the magnitude of local scour. In streams with sand-bed material, the shape of the bed (bed configuration) as described by Richardson et al. may be ripples, dunes, plane bed, or antidunes.[34] The bed configuration depends on the size distribution of the sand-bed material, hydraulic characteristics, and fluid viscosity. The bed configuration may change from dunes to plane bed or antidunes during an increase in flow for a single flood event. It may change back with a decrease in flow. The bed configuration may also change with a change in water temperature or

suspended sediment concentration of silts and clays. The type of bed configuration and change in bed configuration will affect flow velocity, sediment transport, and scour. Richardson et al. discusses bed configuration in detail.[13]

10. Potentially, ice and debris can increase the width of the piers, change the shape of piers and abutments, increase the projected length of an abutment, and cause the flow to plunge downward against the bed. This can increase both local and contraction scour. The magnitude of the increase is still largely undetermined. Debris can be taken into account in the scour equations by estimating how much the debris will increase the width of a pier or length of an abutment. Debris and ice effects on contraction scour can also be accounted for by estimating the amount of flow blockage (decrease in width of the bridge opening) in the equations for contraction scour. Limited field measurements of scour at ice jams indicate the scour can be as much as 3 to 10 m.

2.7 Clear-Water and Live-Bed Scour

There are two conditions for contraction and local scour: clear-water and live-bed scour. Clear-water scour occurs when there is no movement of the bed material in the flow upstream of the crossing or the bed material being transported in the upstream reach is transported in suspension through the scour hole at the pier or abutment at less than the capacity of the flow. At the pier or abutment the acceleration of the flow and vortices created by these obstructions cause the bed material around them to move. Live-bed scour occurs when there is transport of bed material from the upstream reach into the crossing.

Typical clear-water scour situations include

1. coarse-bed material streams,
2. flat gradient streams during low flow,
3. local deposits of larger bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation),
4. armored streambeds where the only locations that tractive forces are adequate to penetrate the armor layer are at piers and/or abutments, and
5. vegetated channels or overbank areas.

During a flood event, bridges over streams with coarse-bed material are often subjected to clear-water scour at low discharges, live-bed scour at the higher discharges and then clear-water scour at the lower discharges on the falling stages. Clear-water scour reaches its maximum over a longer period of time than live-bed scour (Figure 2). This is because clear-water scour occurs mainly in coarse-bed material streams. In fact, local clear-water scour may not reach a maximum until after several floods. Maximum local clear-water pier scour is about 10 percent greater than the equilibrium local live-bed pier scour.

Equation 12 and Equation 13 with $D = D_{50}$, which are used to determine the velocity associated with the initiation of motion, can be used as an indicator for clear-water or live-bed scour conditions. If the mean velocity (V) in the upstream reach is equal to or less than the critical velocity (V_c) of the median diameter (D_{50}) of the bed material, then contraction and local scour will be clear-water scour. Also, if the ratio of the shear velocity of the flow to the fall velocity of the D_{50} of the bed material (V_*/ω) is greater than 3, contraction and local scour may be clear-water (see Section 2.4.2). If the mean velocity is greater than the critical velocity of the median bed material size, live-bed scour will occur.

The preceding technique can be applied to any unvegetated channel or overbank area to determine whether scour is clear-water or live-bed. This procedure should be used with caution for assessing whether or not scour in the overbank will be clear-water or live-bed. For most cases, the presence of vegetation on the overbank will effectively bind and protect the overbank from erosive velocities. Also, in the overbank, generally the velocities before the contraction are small and the bed material so fine that most overbank areas will experience clear-water scour.

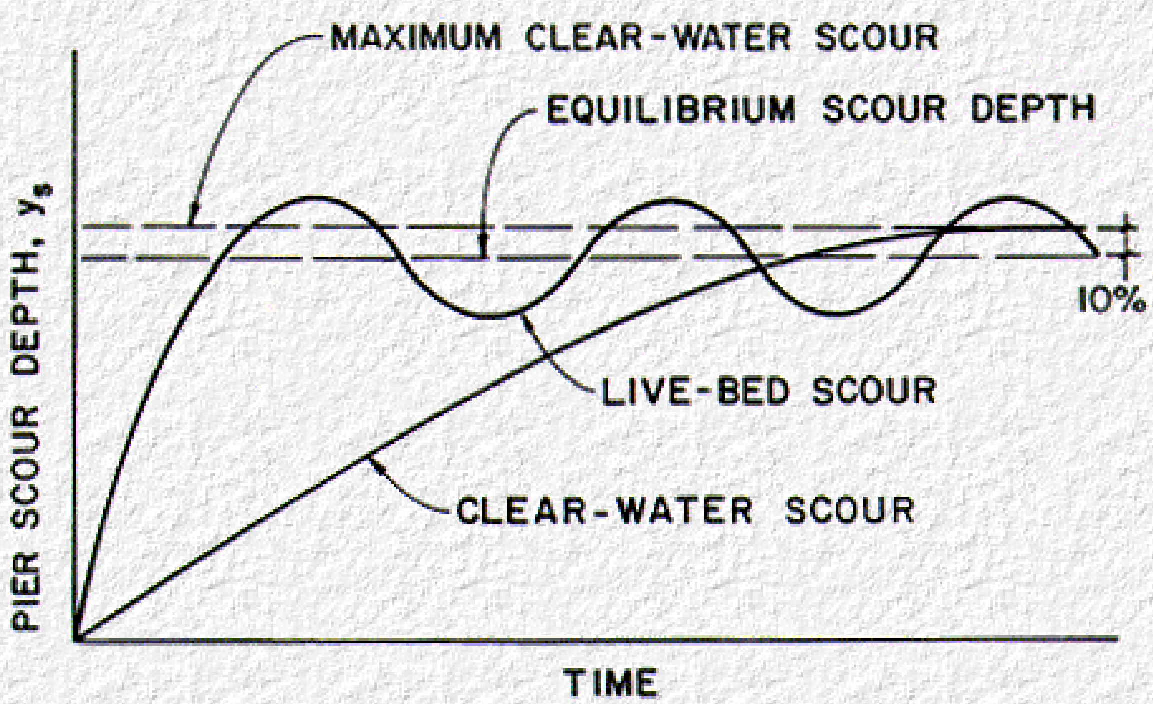

Figure 2. Illustrative Pier Scour Depth in a Sand-bed Stream as a Function of Time (not to scale)

Live-bed pier scour in sand-bed streams with a dune bed configuration fluctuates about the equilibrium scour depth (Figure 2). This is due to the variability of the bed material sediment transport in the approach flow when the bed configuration of the stream is dunes. In this case (dune bed configuration in the channel upstream and through the bridge), maximum depth of pier scour is about 30 percent larger than equilibrium depth of scour. However, with the exception of crossings over large rivers (i.e., the Mississippi, Columbia, etc.), the bed configuration in sand-bed streams will plane out during flood flows due to the increase in velocity and shear stress. For general practice, the maximum depth of pier scour is approximately 10 percent greater than equilibrium scour. This is not illustrated in Figure 2.

For a discussion of bedforms in alluvial channel flow, the reader is referred to Chapter 3 of HIRE.(13) Equations for estimating local scour at abutments or piers are given in Chapter 4 of this document. These equations were developed from laboratory experiments and limited field data for both clear-water and live-bed scour.

2.8 Lateral Shifting of a Stream

Streams are dynamic. Areas of flow concentration continually shift bank lines, and in meandering streams having an "S-shaped" planform, the channel moves both laterally and downstream. A braided stream has numerous channels which are continually changing. In a braided stream, the deepest natural scour occurs when two channels come together or when the flow comes together downstream of an island or bar. This scour depth has been observed to be 1 to 2 times the average flow depth.

A bridge is static. It fixes the stream at one place in time and space. A meandering stream whose channel moves laterally and downstream into the bridge reach can erode the approach embankment and can affect contraction and local scour because of changes in flow direction. A braided stream can shift under a bridge and have two channels come together at a pier or abutment, increasing scour. Descriptions of stream morphology are given in HIRE and HEC-20.(13,12)

Factors that affect lateral shifting of a stream and the stability of a bridge are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, the characteristics of the bed and bank material, and wash load.

It is difficult to anticipate when a change in planform may occur. It may be gradual or the result of a single major flood event. Also, the direction and magnitude of the movement of the stream are not easily predicted. It is difficult to evaluate properly the vulnerability of a bridge due to changes in planform; however, it is important to incorporate potential planform changes into the design of new bridges and design of countermeasures for existing bridges.

Countermeasures for lateral shifting and instability of the stream may include changes in the bridge design, construction of river control works, protection of abutments with riprap, or careful monitoring of the river in a bridge inspection program. **Serious consideration should be given to placing footings/foundations located on floodplains at elevations approximating those located in the main channel.**

Control of lateral shifting requires river training works, bank stabilizing by riprap, and/or guide banks. The design of these works is beyond the scope of this circular. Design methods are given by FHWA, USACOE, and AASHTO.(12,13,35,36,37,38,39,40,41) Of particular importance are "Hydraulic Analyses for the Location and Design of Bridges", HIRE, "Use of Spurs and Guidebanks for Highway Crossings," HEC-20, and HEC-11.(41,13, 37,12,36)

2.9 Pressure Flow Scour

When bridges are overtopped, the flow hydraulics at the bridge are dramatically altered, and contraction and local scour can increase. This topic is discussed in Section 4.3.5.

Go to Chapter 3

Chapter 3 : HEC 18

Designing Bridges to Resist Scour

Go to Chapter 4, Part I

3.1 Design Philosophy and Concepts

Bridges should be designed to withstand the effects of scour from a superflood (a flood exceeding the 100-year flood) with little risk of failing. This requires careful evaluation of the hydraulic, structural, and geotechnical aspects of bridge foundation design.

Guidance in this chapter is based on the following concepts:

1. The foundation should be designed by an interdisciplinary team of engineers with expertise in hydraulic, geotechnical, and structural design.
2. Hydraulic studies of bridge sites are a necessary part of a bridge design. These studies should address both the sizing of the bridge waterway opening and the design of the foundations to be safe from scour. The scope of the analysis should be commensurate with the importance of the highway and consequences of failure.
3. Consideration must be given to the limitations and gaps in existing knowledge when using currently available formulas for estimating scour. **The designer needs to apply engineering judgment in comparing results obtained from scour computations with available hydrologic and hydraulic data to achieve a reasonable and prudent design.** Such data should include:
   . Performance of existing structures during past floods,
   b. Effects of regulation and control of flood discharges,
   c. Hydrologic characteristics and flood history of the stream and similar streams, and
   d. Whether the bridge is structurally continuous.
4. The principles of economic analysis and experience with actual flood damage indicate that it is almost always cost-effective to provide a foundation that will not fail, even from a very large flood event or superflood. Generally, occasional damage to highway approaches from rare floods can be repaired quickly to restore traffic service. On the other hand, a bridge which collapses or suffers major structural damage from scour can create safety hazards to motorists as well as significant social impacts and economic losses over a long period of time. Aside from the costs to the highway agency of replacing or repairing the bridge and constructing and maintaining detours, there can be significant costs to communities or entire regions due to additional detour travel time, inconvenience, and lost business opportunities. Therefore, a higher hydraulic standard is warranted for the design of bridge foundations to resist scour than is usually required for sizing of the bridge waterway. This concept is reflected in the following design procedure.

3.2 General Design Procedure

The general design procedure for scour outlined in the following steps is recommended for determining bridge type, size, and location (TS&L) of substructure units:

Step 1. Select the flood event(s) that are expected to produce the most severe scour conditions. Experience indicates that this is likely to be the 100-year flood or the overtopping flood when it is less than the 100-year flood. Check the 100-year flood or the overtopping flood (if less than the 100-year flood) and other flood events if there is evidence that such events would create deeper scour than the 100-year or overtopping floods. Overtopping refers to flow over the approach embankment(s), the bridge itself, or both.

Step 2. Develop water surface profiles for the flood flows in step 1, taking care to evaluate the range of potential tailwater conditions below the bridge which could occur during these floods. The FHWA microcomputer software WSPRO, is recommended for this task.[24] The Corps of Engineers HEC-2 program or the new HEC River Analysis System (RAS) can also be used.[42,89]

Step 3. Using the seven-step Specific Design Approach in Chapter 4, estimate total scour for the worst condition from steps 1 and 2 above. The resulting scour from the selected flood event should be considered in the design of a foundation. For this condition, minimum geotechnical safety factors commonly accepted by State highway agencies should be applied. For example, for pile design in friction, a commonly applied factor of safety ranges from 2 to 3, for the 100-year or overtopping flood.

Step 4. Plot the total scour depths obtained in step 3 on a cross section of the stream channel and floodplain at the bridge site.

Step 5. Evaluate the results obtained in steps 3 and 4. Are they reasonable, considering the limitations in current scour estimating procedures? The scour depth(s) adopted may differ from the equation value(s) based on engineering judgment.

Step 6. Evaluate the bridge TS&L on the basis of the scour analysis performed in steps 3 through 5. Modify the TS&L as necessary.

a. Visualize the overall flood flow pattern at the bridge site for the design conditions. Use this mental picture to identify those bridge elements most vulnerable to flood flows and resulting scour.

b. The extent of protection to be provided should be determined by:

- The degree of uncertainty in the scour prediction method.

- The potential for and consequences of failure.
- The added cost of making the bridge less vulnerable to scour. Design measures incorporated in the original construction are almost always less costly than retrofitting scour countermeasures.

Step 7. Perform the bridge foundation analysis on the basis that all streambed material in the scour prism above the total scour line (step 4) has been removed and is not available for bearing or lateral support. All foundations should be designed in accordance with the AASHTO Standard Specifications for Highway Bridges.(43) In the case of a pile foundation, the piling should be designed for additional lateral restraint and column action because of the increase in unsupported pile length after scour. In areas where the local scour is confined to the proximity of the footing, the lateral ground stresses on the pile length which remains embedded may not be significantly reduced from the pre-local scour conditions. The depth of local scour and volume of soil removed from above the pile group should be considered by geotechnical engineers when computing pile embedment to sustain vertical load.

a. <u>Spread Footings On Soil</u>

- Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration.
- Place the bottom of the footing below the total scour line from step 4.
- The top of the footing can act as a local scour arrestor.

b. <u>Spread Footings On Rock Highly Resistant To Scour</u>

Place the bottom of the footing directly on the cleaned rock surface for massive rock formations (such as granite) that are highly resistant to scour. Small embedments (keying) should be avoided since blasting to achieve keying frequently damages the sub-footing rock structure and makes it more susceptible to scour. If footings on smooth massive rock surfaces require lateral constraint, steel dowels should be drilled and grouted into the rock below the footing level.

c. <u>Spread Footings On Erodible Rock</u>

Weathered or other potentially erodible rock formations need to be carefully assessed for scour. An engineering geologist familiar with the area geology should be consulted to determine if rock or soil or other criteria should be used to calculate the support for the spread footing foundation. The decision should be based on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated structure life. An important consideration may be the existence of a high quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth should be estimated (steps 4 and 5) and the footing base placed below that depth. Excavation into weathered rock should be made with care. If blasting is required, light, closely spaced charges should be used to minimize overbreak beneath the footing level. Loose rock pieces should be removed and the zone filled with clean concrete. In any event, the final footing should be poured in contact with the sides of the excavation for the full designed footing thickness to minimize water intrusion below footing level. Guidance on scourability of rock formations is given in FHWA memorandum "Scourability of Rock Formations" dated July 19, 1991.

d. Spread Footings Placed On Tremie Seals And Supported On Soil

- Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration.
- Place the bottom of the footing below the total scour line from step 4.

e. For Deep Foundations (Drilled Shaft And Driven Piling) With Footings Or Caps

Placing the top of the footing or pile cap below the streambed a depth equal to the estimated long-term degradation and contraction scour depth will minimize obstruction to flood flows and resulting local scour. Even lower footing elevations may be desirable for pile supported footings when the piles could be damaged by erosion and corrosion from exposure to river or tidal currents. For more discussion on pile and drilled shaft foundations, see the manuals on Design and Construction of Driven Pile Foundations and Drilled Shafts.[87,88]

f. Stub Abutments on Piling

Stub abutments positioned in the embankment should be founded on piling driven below the elevation of the thalweg in the bridge waterway to assure structural integrity in the event the thalweg shifts and the bed material around the piling scours to the thalweg elevation.

Step 8. Repeat the procedure in steps 2 through 6 above and calculate the scour for a superflood. It is recommended that this superflood (or check flood) be on the order of a 500-year event. If the magnitude of the 500-year flood is not available from a published source, use a discharge equal to 1.7 x Q_{100}. However, flows greater or less than these suggested floods may be appropriate depending upon hydrologic considerations and the consequences associated with damage to the bridge. An overtopping flood less than the 500-year flood may produce the worst-case situation for checking the foundation design. The foundation design determined under step 7 should be reevaluated for the superflood condition and design modifications made where required.

a. Check to make sure that the bottom of spread footings on soil or weathered rock is below the total scour depth for the superflood.

b. **All foundations should have a minimum factor of safety of 1.0 (ultimate load) under the superflood conditions.** Note that in actual practice, the calculations for step 8 would be performed concurrently with steps 1 through 7 for efficiency of operation.

3.3 Checklist of Design Considerations

3.3.1 General

1. Raise the bridge superstructure elevation above the general elevation of the approach roadways wherever practicable. This provides for overtopping of approach embankments and relief from the hydraulic forces acting at the bridge. This is particularly important for streams carrying large amounts of debris which could clog the waterway at the bridge.

It is recommended that the elevation of the lower cord of the bridge be increased a minimum of 0.6 m above the normal freeboard for the 100-year flood for streams that carry a large amount of debris.

2. Superstructures should be securely anchored to the substructure if buoyant, or if debris and ice forces are probable. Further, the superstructure should be shallow and open to minimize resistance to the flow where overtopping is likely.

3. Continuous span bridges withstand forces due to scour and resultant foundation movement better than simple span bridges. Continuous spans provide alternate load paths (redundancy) for unbalanced forces caused by settlement and/or rotation of the foundations. This type of structural design is recommended for bridges where there is a significant scour potential.

4. Local scour holes at piers and abutments may overlap one another in some instances. If local scour holes do overlap, the scour is indeterminate and is deeper. The topwidth of a local scour hole on each side of the pier ranges from 1.0 to 2.8 times the depth of scour. A topwidth value of 2.0 times the depth of scour is suggested for practical applications.

5. For pile and drilled shaft supported substructures subjected to scour, a reevaluation of the foundation design may require a change in the pile or shaft length, number, cross-sectional dimension and type based on the loading and performance requirements and site-specific conditions.

6. At some bridge sites, hydraulics and traffic conditions may necessitate consideration of a bridge that will be partially or even totally inundated during high flows. This consideration results in pressure flow through the bridge waterway.

Section 4.3.5 has a discussion on pressure flow scour for these cases.

3.3.2 Piers

1. Pier foundations on floodplains should be designed to the same elevation as pier foundations in the stream channel if there is a likelihood that the channel will shift its location over the life of the bridge.

2. Align piers with the direction of flood flows. Assess the hydraulic advantages of round piers, particularly where there are complex flow patterns during flood events.

3. Streamline piers to decrease scour and minimize potential for buildup of ice and debris. Use ice and debris deflectors where appropriate.

4. Evaluate the hazards of ice and debris buildup when considering use of multiple pile bents in stream channels. Where ice and debris buildup is a problem, consider the bent a solid pier for purposes of estimating scour. Consider use of other pier types where clogging of the waterway area could be a major problem.

3.3.3 Abutments

1. The equations used to estimate the magnitude of abutment scour were developed in a laboratory under ideal conditions and for the most part lack field verification. Because conditions in the field are different from those in the laboratory, these equations may tend to over predict the magnitude of scour that may be expected to develop. Recognizing this, it is recommended that the abutment scour equations be used to develop insight as to the scour potential at an abutment. If the engineer desires, the abutment may be designed to resist the computed scour. As an alternate, riprap and guide banks can be used to protect the abutment from failure. Riprap or some other protection should always be used to protect the abutment from erosion. Proper design techniques and placement procedures for rock riprap and guide banks are discussed in Section 7.5 and HEC-20.(12)

2. Relief bridges, guide banks, and river training works should be used, where needed, to minimize the effects of adverse flow conditions at abutments.

3. Where ice build-up is likely to be a problem, set the toe of spill-through slopes or vertical abutments back from the edge of the channel bank to facilitate passage of the ice.

4. Wherever possible, use spill-through (sloping) abutments. Scour at spill-through abutments is about 50 percent of that of vertical wall abutments.

Go to Chapter 4, Part I

Chapter 4 : HEC 18

Estimating Scour at Bridges
Part I

Go to Chapter 4, Part II

4.1 Introduction

This chapter presents the methods and equations for determining total scour at a bridge, i.e., long-term aggradation or degradation, contraction scour, and local scour. Example problems are given for both riverine and tidal conditions at the end of the chapter. While the scour equations presented are based on riverine conditions, they can be used for tidal waterways. Section 4.5 discusses hydrodynamics and scour methodologies for tidal waterways.

Prior to applying the various scour estimating methods for contraction and local scour, it is necessary to:

1. obtain the fixed-bed channel hydraulics,
2. determine the long-term impact of degradation or aggradation on the bed profile,
3. if degradation occurs, adjust the fixed-bed hydraulics to reflect this change; if aggradation occurs, see Section 7.5.3 for guidance, and
4. compute the bridge hydraulics.

4.2 Specific Design Approach

The seven steps recommended for estimating scour at bridges are:

- Step 1: Determine scour analysis variables.
- Step 2: Analyze long-term bed elevation change.
- Step 3: Evaluate the scour analysis method.
- Step 4: Compute the magnitude of contraction scour.
- Step 5: Compute the magnitude of local scour at piers.
- Step 6: Compute the magnitude of local scour at abutments or place the abutment foundation as described in Section 3.3.3.
- Step 7: Plot and evaluate the total scour depths as outlined in Steps 4 through 6 of the General Design Procedure in Chapter 3.

The engineer should evaluate how reasonable the individual estimates of contraction and local scour depths are in steps 4 and 5 and evaluate the reasonableness of the total scour in step 7. The results from this Specific Design Approach complete steps 1 through 6 of Chapter 3. The design must now proceed to steps 7 and 8 of the General Design Procedure in Chapter 3.

The procedures for each of the steps, including recommended scour equations, are discussed in detail in the following sections.

4.3 Detailed Procedures

4.3.1 Step 1: Determine Scour Analysis Variables

1. Determine the magnitude of the discharges for the floods in steps 1 and 8 of the General Design Procedure in Chapter 3, including the overtopping flood when applicable. If the magnitude of the 500-year flood is not available from a published source, use a discharge equal to 1.7 times the Q_{100}. Experience has shown that the incipient overtopping discharge often puts the most stress on a bridge.

However, special conditions (angle of attack, pressure flow, decrease in velocity or discharge resulting from high flows overtopping approaches or going through relief bridges, ice jams, etc.) may cause a more severe condition for scour with a flow smaller than the overtopping or 100-year flood.

2. Determine if there are existing or potential future factors that will produce a combination of high discharge and low tailwater control. Are there bedrock or other controls (old diversion structures, erosion control checks, other bridges, etc.) that might be lowered or removed? Are there dams or locks downstream that would control the tailwater elevation seasonally? Are there dams upstream or downstream that could control the elevation of the water surface at the bridge? Select the lowest reasonable downstream water-surface elevation and the largest discharge to estimate the greatest scour potential. Assess the distribution of the velocity and discharge per foot of width for the design flow and other flows through the bridge opening. Also, consider the contraction and expansion of the flow in the bridge waterway, as well as present conditions and anticipated future changes in the river.

3. Determine the water-surface profiles for the discharges judged to produce the most scour from step 1, using WSPRO, HEC-2, or the new HEC River Analysis System (RAS).[24,42,89] In some instances, the designer may wish to use BRI-STARS.[14] Hydraulic studies by the USACOE, USGS, the Federal Emergency Management Agency (FEMA), etc. are potentially useful sources of hydraulic data to calibrate, verify, and evaluate results from WSPRO or HEC-2. The engineer should anticipate future conditions at the bridge, in the upstream watershed, and at downstream water-surface elevation controls as outlined in HEC-20.[12] From computer analysis and from other hydraulic studies, determine input variables such as the discharge, velocity and depth needed for the scour calculations.

4. Collect and summarize the following information as appropriate (see HEC-20 for a step-wise analysis procedure).[12]

   . Boring logs to define geologic substrata at the bridge site.
   b. Bed material size, gradation, and distribution in the bridge reach.
   c. Existing stream and floodplain cross section through the reach.
   d. Stream platform.
   e. Watershed characteristics.
   f. Scour data on other bridges in the area.
   g. Slope of energy grade line upstream and downstream of the bridge.
   h. History of flooding.
   i. Location of bridge site with respect to other bridges in the area, confluence with tributaries close to the site, bed rock controls, man-made controls (dams, old check structures, river training works, etc.), and confluence with another stream downstream.
   j. Character of the stream (perennial, flashy, intermittent, gradual peaks, etc.).

k. Geomorphology of the site (floodplain stream; crossing of a delta, youthful, mature or old age stream; crossing of an alluvial fan; meandering, straight or braided stream; etc.).
l. Erosion history of the stream.
m. Development history (consider present and future conditions) of the stream and watershed. Collect maps, ground photographs, aerial photographs; interview local residents; check for water resource projects planned or contemplated.
n. Sand and gravel mining from the streambed or floodplain up- and downstream from site.
o. Other factors that could affect the bridge.
p. Make a qualitative evaluation of the site with an estimate of the potential for stream movement and its effect on the bridge.

4.3.2 Step 2: Analysis of Long-Term Bed Elevation Change

1. Using the information collected in step 1 above, determine qualitatively the long-term trend in the streambed elevation. The USACOE, USGS, and other agencies may have information on historic and current streambed elevations. Where conditions indicate that significant aggradation or degradation is likely, estimate the change in bed elevation over the next 100 years using one or more of the following:

. Straight line extrapolation of present trends,
b. Engineering judgment,
c. The worst-case scenarios. For example, in the case of a confluence with another stream just downstream of the bridge, assume the design flood would occur with a low downstream water-surface elevation through a qualitative assessment of flood magnitudes and river conditions on the main stream and its tributary,
d. Available sediment routing or sediment continuity computer programs such as BRI-STARS and the Corps of Engineers HEC-6.(14,15)

2. If the stream is aggrading and this condition can be expected to affect the crossing, taking into account contraction scour, consider relocating the bridge or raising the low cord of the bridge. With an aggrading stream, use the present streambed elevation as the baseline for scour estimates because a major flood can occur and reverse the aggradational trend.

3. If the stream is degrading, use an estimate of the change in elevation in the calculations of total scour.

4.3.3 Step 3: Evaluate the Scour Analysis Method

The recommended method is based on the assumption that the scour components develop independently. Thus, the potential local scour is added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics.

1. Determine the natural channel hydraulics for a fixed-bed condition based on existing conditions,

2. Assess the expected profile and platform changes,

3. Adjust the fixed-bed hydraulics to reflect any expected long-term profile or platform changes,

4. Compute contraction scour using either the live-bed or clear-water contraction scour equations or both (see step 4 below),

5. Compute local scour using the adjusted fixed-bed channel hydraulics to reflect any expected long-term profile or platform change (see steps 5 and 6 below), and

6. Add the three scour components (long-term degradation, contraction scour and local scour) to obtain the total scour (see

Chapter 3, General Design Procedure, step 4, or Chapter 4, step 7 of the Specific Design Procedure).

4.3.4 Step 4: Compute the Magnitude of Contraction Scour

General. Contraction scour at bridge sites can be broken down into four conditions (cases) depending on the type of contraction, and whether there is overbank flow or relief bridges. Regardless of the case, contraction scour can be evaluated using two basic equations: (1) live-bed scour, and (2) clear-water scour. For any case or condition, it is only necessary to determine if the flow in the main channel or overbank area upstream of the bridge, or approaching a relief bridge, is transporting bed material (live-bed) or is not (clear-water), and then apply the appropriate equation with the variables defined according to the location of contraction scour (channel or overbank). **Live-bed scour depths may be limited if there are appreciable amounts of large-sized particles in the bed material. It is appropriate, then, to use the clear-water scour equation and use the lessor of the two depths. Also, it is appropriate to use the clear-water scour equation if the transport of bed material from upstream of the contraction is small in quantity or composed of fine material that washes through the contraction in suspension.**

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity use the equation derived in Chapter 2. This equation is reiterated as follows:

$$V_c = \frac{K_s^{1/2}(S_s - 1)^{1/2}\ D^{1/2}\ y^{1/6}}{n} \qquad (15)$$

Using $K_s = 0.039$, $S_s = 2.65$, and $n = 0.041\ D^{1/6}$, Equation 15 for critical velocity V_c becomes:

$$V_c = 6.19\ y^{1/6}\ D_{50}^{1/3} \qquad (16)$$

where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, m/s
K_s = Shields parameter
S_s = Specific gravity of the bed material
y = Depth of flow, m
D = Particle size for V_c, m
D_{50} = Particle size in a mixture of which 50% are smaller, m
n = Manning's roughness coefficient

Contraction Scour Conditions. Four conditions (cases) of contraction scour are commonly encountered:

Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:

- The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river;

b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment; or
c. Abutments are set back from the stream channel.

Case 2. Flow is confined to the main channel (i.e., there is no overbank flow). The normal river channel width becomes narrower due to the bridge itself or the bridge site is located at a narrower reach of the river.

Case 3. A relief bridge in the overbank area with little or no bed material transport in the overbank area (i.e., clear-water scour).

Case 4. A relief bridge over a secondary stream in the overbank area with bed material transport (similar to case 1).

Notes:

1. Cases 1, 2, and 4 may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows. To determine if there is bed material transport compute the critical velocity at the approach section for the D_{50} of the bed material using the equation given above and compare to the mean velocity at the approach section. To determine if the bed material will be washed through the contraction determine the ratio of the shear velocity (V_*) in the contracted section to the fall velocity (ω) of the (D_{50}) of the bed material being transported from the upstream reach (see the definition of V_* in the live-bed contraction scour equation). If the ratio is much larger than 3, then the bed material from the upstream reach will be mostly suspended bed material discharge and may wash through the contracted reach (clear-water scour).

2. **Case 1c is very complex.** The depth of contraction scour depends on factors such as (1) how far back from the bank line the abutment is set, (2) the condition of the overbank (is it easily eroded, are there trees on the bank, is it a high bank, etc.), (3) whether the stream is narrower or wider at the bridge than at the upstream section, (4) the magnitude of the overbank flow that is returned to the bridge opening, and (5) the distribution of the flow in the bridge section, and (6) other factors.

The main channel under the bridge may be live-bed scour; whereas, the set-back overbank area may be clear-water scour.

WSPRO can be used to determine the distribution of flow between the main channel and the set-back overbank areas in the contracted bridge opening.[24]

If the abutment is set back only a small distance from the bank (less than 3 to 5 times the depth of flow through the bridge), there is the possibility that the combination of contraction scour and abutment scour may destroy the bank. Also, the two scour mechanisms are not independent. Consideration should be given to using a guide bank and/or protecting the bank and bed under the bridge in the overflow area with rock riprap. See Chapter 7 for guidance on designing rock riprap.

3. Case 3 may be clear-water scour even though the floodplain bed material is composed of fine sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are (1) there may be vegetation growing part of the year, and (2) the fine bed material may go into suspension (wash load) at the bridge and not influence the contraction scour.

4. **Case 4** is similar to Case 3, but there is sediment transport into the relief bridge opening (live-bed scour). This case can occur when a relief bridge is over a secondary channel on the floodplain. Hydraulically this is no different from case 1, but analysis is required to determine the floodplain discharge associated with the relief opening and the flow distribution going to and through the relief bridge. This information could be obtained from WSPRO.[24]

Live-Bed Contraction Scour. A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section.[16] The original equation is given in Chapter 2. The modification is to eliminate the ratio of Manning's n (see the following Note #3). The equation assumes that bed material is being transported in the upstream section.

$$\frac{y_1}{y_2} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \qquad (17)$$

$$y_s = y_2 - y_o = \text{(average scour depth)} \qquad (18)$$

where:

y_1 = Average depth in the upstream main channel, m
y_2 = Average depth in the contracted section, m
y_o = Existing depth in the contracted section before scour, m (see note 7, Section 4.3.4.)
Q_1 = Flow in the upstream channel transporting sediment, m^3/s
Q_2 = Flow in the contracted channel, m^3/s
W_1 = Bottom width of the upstream main channel, m
W_2 = Bottom width of the main channel in the contracted section less pier width(s), m
k_1 = Exponent determined below

V_*/ω	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

V_* = $(\tau_o/\rho)^{1/2} = (gy_1 S_1)^{1/2}$, shear velocity in the upstream section, m/s
ω = Fall velocity of bed material based on the D_{50}, m/s (see Figure 3)
g = Acceleration of gravity (9.81 m/s^2)
S_1 = Slope of energy grade line of main channel, m/m
τ_o = Shear stress on the bed, Pa (N/m^2)
ρ = Density of water (1000 kg/m^3)

Notes:

1. Q_2 may be the total flow going through the bridge opening as in cases 1a and 1b. It is not the total flow for case 1c.

2. Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.

3. The Manning's n ratio (see Equation 1) can be significant for a condition of dune bed in the main channel and a corresponding plane bed, washed out dunes or antidunes in the contracted channel. However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planing out (which decreases resistance to flow, increases the velocity and the transport of bed material at the bridge). That is, Laursen's equation indicates a decrease in scour for this case, whereas in reality, there would be an increase in scour depth. In addition, at flood flows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning's n will be equal. Consequently, the n value ratio is not recommended or presented in the recommended Equation 17.

4. W_1 and W_2 are not always easily defined. In some cases, it is acceptable to use the top width of the main channel to define these widths. Whether top width or bottom width is used, it is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.

5. The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.

6. Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.

7. In sand channel streams where the contraction scour hole is filled in on the falling stage, the y_1 depth may be appropriate. Sketches or surveys through the bridge can help in determining the existing bed elevation.

8. **Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation (given in the next section) in addition to the live-bed equation, and that the smaller calculated scour depth be used.**

Clear-Water Contraction Scour. The recommended clear-water contraction scour equation is based on a development suggested by Laursen (presented in Chapter 2).[17] The equation is:

$$y_2 = \left[\frac{n^2\,Q^2}{K_s(S_s-1)\,D_m\,W_2}\right]^{3/7} \qquad (19)$$

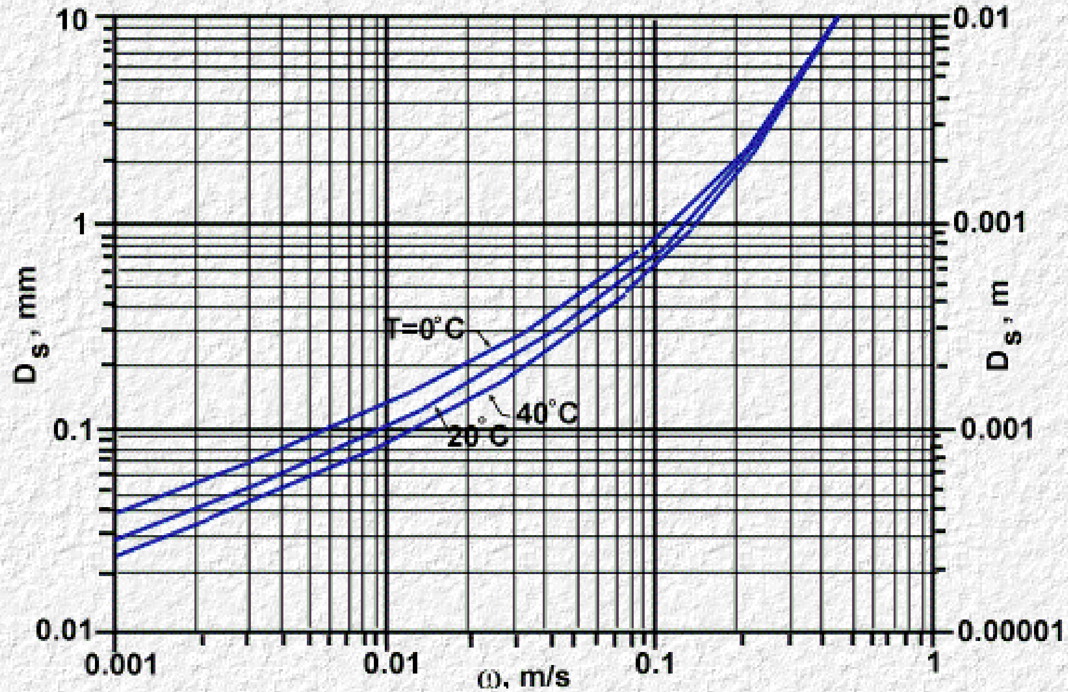

Figure 3. Fall Velocity of Sand-sized Particles

With Manning's n given by Strickler's in metric form as $n = 0.040\,D_m^{1/6}$, $S_s = 2.65$, and Shields Coefficient (K_s) = 0.039 (Chapter 2) the equation is:

$$y_2 = \left[\frac{0.025\,Q^2}{D_m^{2/3}\,W^2}\right]^{3/7} \qquad (20)$$

$y_s = y_2 - y_o$ = (average scour depth, m) (20a)

where:

y_2 = Average depth in the contracted section after contraction scour, m
n = Manning's roughness coefficient
Q = Discharge through the bridge or on the overbank at the bridge associated with the width W, m^3/s
K_s = Shield's coefficient
S_s = Specific gravity (2.65 for quartz)
D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, m
D_{50} = Median diameter of bed material, m
W = Bottom width of the contracted section less pier widths, m
y_o = Existing depth in the contracted section before scour, m

For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

To obtain the distribution of the scour depths across a section (say at or downstream of a bend) use WSPRO to obtain the velocity of each stream tube and Equation 7 or Equation 9 in Chapter 2 to obtain the scour depth in each stream tube. Changes in bed material size across a stream can be accounted for by this method.

Other Contraction Scour Conditions. Contraction scour resulting from variable water surfaces downstream of the bridge is analyzed by determining the lowest potential water-surface elevation downstream of the bridge insofar as scour processes are concerned. Use the WSPRO computer program to determine the flow variables, such as velocity and depths, through the bridge.(24) With these variables, determine contraction and local scour depths.

Contraction scour in a channel bendway resulting from the flow through the bridge being concentrated toward the outside of the bend is analyzed by determining the superelevation of the water surface on the outside of the bend and estimating the resulting velocities and depths through the bridge. The maximum velocity in the outer part of the bend can be 1.5 to 2 times the mean velocity. A physical model study can also be used to determine the velocity and scour depth distribution through the bridge for this case.

Estimating contraction scour for unusual situations involves particular skills in the application of principles of river mechanics to the site-specific conditions. Such studies should be undertaken by engineers experienced in the fields of hydraulics and river mechanics.

Backwater. The **live-bed** contraction scour equation is derived assuming a uniform reach above and a long contraction into a uniform reach below the bridge. With **live-bed scour** the equation computes a depth after the long contraction where the sediment transport in the downstream reach is equal to the sediment transport out. The **clear-water** contraction scour equations are derived assuming that the depth at the bridge increases until the shear-stress and velocity are decreased so that there is no longer any sediment transport. With the **clear-water** equations it is assumed that flow goes from one uniform flow condition to another. Both Equations calculate contraction scour depth assuming a level water surface ($y_s = y_2 - y_1$). A more consistent computation would be to write an energy balance before and after the scour. For **live-bed** the energy balance would be between the approach section (1) and the contracted section (2). Whereas, for **clear-water** scour it would be the energy at the same section before (1) and after (2) the contraction scour.

As explained in Chapter 2 (Section 2.5) normally, except for the difference in depths the other terms in the energy equation are small.

Backwater, in extreme cases, can decrease the velocity, shear stress and the sediment transport in the upstream section. This will increase the scour at the contracted section. The backwater can, by storing sediment in the upstream section change live-bed scour to clear-water scour.

For additional discussion of backwater, see Section 2.5.

4.3.5 Step 5: Compute the Magnitude of Local Scour at Piers

General. Local scour at piers is a function of bed material size, flow characteristics, fluid properties and the geometry of the pier. The subject has been studied extensively in the laboratory, but there is limited field data. As a result of the many studies, there are many equations. In general, the equations are for live-bed scour in cohesionless sand-bed streams.

A graphical comparison of the more common equations is given in Figure 4 and Figure 5.[44] An equation given by Melville and Sutherland to calculate scour depths for live-bed scour in sand-bed streams has been added to the original figures.[29] Some of the equations have velocity as a variable, normally in the form of a Froude Number. However, some equations, such as Laursen's do not include velocity.[16] A Froude Number of 0.3 was used in Figure 4 for purposes of comparing commonly used scour equations. In Figure 5, the equations are compared with some field data measurements. As can be seen from Figure 5, the Colorado State University (CSU) equation includes all the points, but gives lower values of scour than the Jain and Fischer, Laursen, Melville and Sutherland, and Neill equations.[13,45,46,29,44] The CSU equation includes the velocity of the flow just upstream of the pier by including the Froude Number in the equation. Chang pointed out that Laursen's 1960 equation is essentially a special case of the CSU equation with the Fr = 0.4 (see Figure 6).[47]

The equations illustrated in Figure 4, Figure 5, and Figure 6 do not take into account the possibility that larger sizes in the bed material could armor the scour hole. That is, the large sizes in the bed material may at some depth of scour limit the scour depth. Raudkivi, Melville and Sutherland, and others developed equations based on laboratory and limited field data which take into consideration large particles in the bed.[27,29,26] Most of the field scour depths were measured after the flood had occurred and the depths were not representative of the flow conditions that caused them. **Therefore, these equations are not recommended for use.**

In Figure 6, the relationship between y_s/a and y_1/a from the CSU equation is given as a function of the Froude Number. This relation was developed by Chang.[47] Note that Laursen's pier scour equation is a special case of the CSU equation when the Froude Number is 0.4. Values of y_s/a around 3.0 were obtained by Jain and Fischer for chute-and-pool flows with Froude Numbers as high as 1.5.[45] The largest value of y_s/a for antidune flow was 2.5 with a Froude Number of 1.2. Thus, the CSU equation will correctly predict scour depths for upper regime flows (plane bed, antidunes, and chutes and pools).[47]

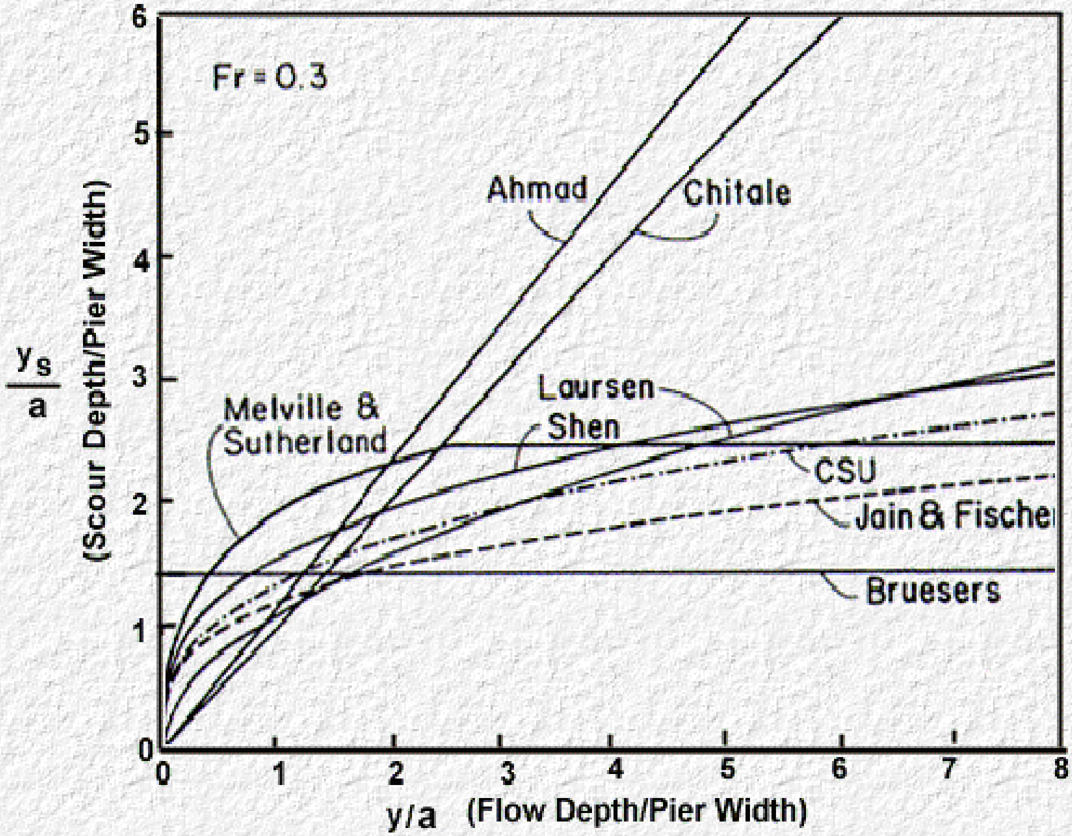

Figure 4. Comparison of Scour Equations for Variable Depth Ratios (y/a) (after Jones)[44]

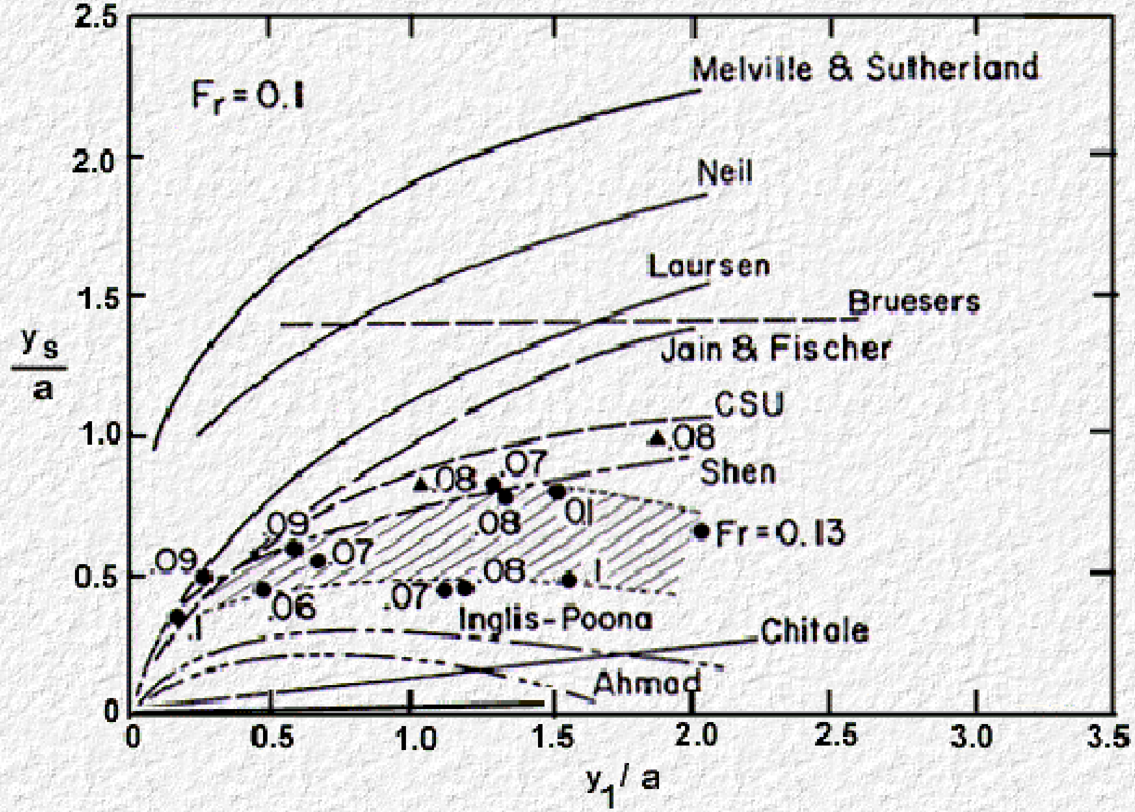

Figure 5. Comparison of Scour Equations with Field Scour Measurements (after Jones)[44]

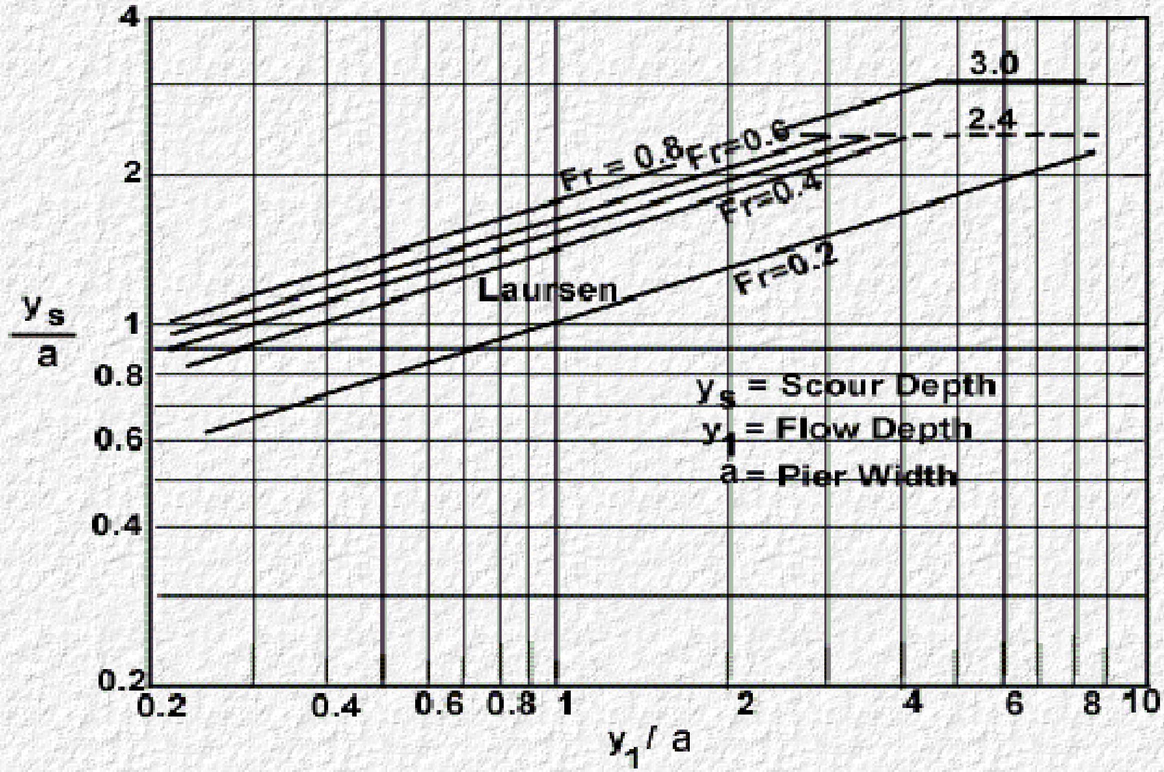

Figure 6. Values of y_s/a versus y_1/a for CSU's Equation 47

Chang noted that in all the data he studied, there were no values of the ratio of scour depth to pier width (y_s/a) larger than 2.3.(47) From laboratory data, Melville and Sutherland reported 2.4 as an upper limit ratio for cylindrical piers.(29) In these studies, the Froude Number was less than 1.0. These upper limits were derived for circular piers and were uncorrected for pier shape and for skew. Also, pressure flow or debris can increase the ratio.

From the above discussion, the ratio of y_s/a can be as large as 3 at large Froude Numbers. Therefore, it is recommended that the maximum value of the ratio be taken as 2.4 for Froude Numbers less than or equal to 0.8 and 3.0 for larger Froude Numbers. These limiting ratio values apply only to round nose piers which are aligned with the flow.

Computing Pier Scour. **To determine pier scour, an equation based on the CSU equation is recommended for both live-bed and clear-water pier scour.**(13) The equation predicts maximum pier scour depths. The equation is:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3\ K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \qquad (21)$$

For round nose piers aligned with the flow:

$y_s \leq 2.4$ times the pier width (a) for $Fr \leq 0.8$ (21a)

$y_s \leq 3.0$ times the pier width (a) for $Fr > 0.8$

In terms of y_s/a, Equation 21 is:

$$\frac{y_s}{a} = 2.0\ K_1\ K_2\ K_3\ K_4 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \qquad (22)$$

where:

- y_s = Scour depth, m
- y_1 = Flow depth directly upstream of the pier, m
- K_1 = Correction factor for pier nose shape from Figure 7 and Table 2
- K_2 = Correction factor for angle of attack of flow from Table 3 or Equation 23
- K_3 = Correction factor for bed condition from Table 4
- K_4 = Correction factor for armoring by bed material size from Equation 24 and Table 5
- a = Pier width, m
- L = Length of pier, m
- Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
- V_1 = Mean velocity of flow directly upstream of the pier, m/s
- g = Acceleration of gravity (9.81 m/s^2)

The correction factor for angle of attack of the flow K_2 given in Table 3 can be calculated using the following equation:

$$K_2 = (\cos\theta + L/a \sin\theta)^{0.65} \qquad (23)$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 23 and Table 3.

Table 2. Correction Factor, K_1, for Pier Nose Shape

Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 3. Correction Factor, K_2, for Angle of Attack, θ, of the Flow

Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5

30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier, m			

Table 4. Increase in Equilibrium Pier Scour Depths (K_3) for Bed Condition

Bed Condition	Dune Height m	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	3> H ≥ 0.6	1.1
Medium Dunes	9> H ≥ 3	1.2 to 1.1
Large Dunes	H ≥ 9	1.3

The correction factor K_4 decreases scour depths for armoring of the scour hole for bed materials that have a D_{50} equal to or larger than 0.06 m ($D_{50} \geq 0.06$ m). The correction factor results from recent research for FHWA by Molinas at CSU which showed that when the approach velocity (V_1) is less than the critical velocity (V_{c90}) of the D_{90} size of the bed material and there is a gradation in sizes in the bed material, the D_{90} will limit the scour depth.(31,32) The equation developed by Jones from analysis of the data is:(25)

$$K_4 = [1 - 0.89(1 - V_R)^2]^{0.5} \quad (24)$$

where:

$$V_R = \left[\frac{V_1 - V_i}{V_{c90} - V_i}\right] \quad (24a)$$

$$V_i = 0.645\left[\frac{D_{50}}{a}\right]^{0.053} V_{c50} \quad (24b)$$

V_R = Velocity ratio
V_1 = Approach velocity, m/s
V_i = Approach velocity when particles at a pier begin to move, m/s
V_{c90} = Critical velocity for D_{90} bed material size, m/s
V_{c50} = Critical velocity for D_{50} bed material size, m/s
a = Pier width, m

$$V_c = 6.19\, y^{1/6}\, D_c^{1/3} \quad (24c)$$

D_c = Critical particle size for the critical velocity V_c, m

Limiting K_4 values and bed material size are given in Table 5.

Table 5. Limits for Bed Material Size and K_4 Values

Factor	Minimum Bed Material Size	Minimum K_4 Value	$V_R > 1.0$
K_4	$D_{50} \geq 0.06$ m	0.7	1.0

1. The correction factor K_1 for pier nose shape should be determined using Table 2 for angles of attack up to 5 degrees. **For greater angles, K_2 dominates and K_1 should be considered as 1.0.** If L/a is larger than 12, use the values for L/a = 12 as a maximum in Table 3 and Equation 24.

2. The values of the correction factor K_2 should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor directly from the table will result in a significant over-prediction of scour if (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the K_2 factor by selecting the effective length of the pier actually subjected to the angle of attack of the flow.

3. The correction factor K_3 results from the fact that for plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed with Equation 21. In the unusual situation where a dune bed configuration with large dunes exists at a site during flood flow, the maximum pier scour may be 30 percent greater than the predicted equation value. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune bed configuration at flood flow, the dunes will be smaller and the maximum scour may be only 10 to 20 percent larger than equilibrium scour. For antidune bed configuration the maximum scour depth may be 10 percent greater than the computed equilibrium pier scour depth.

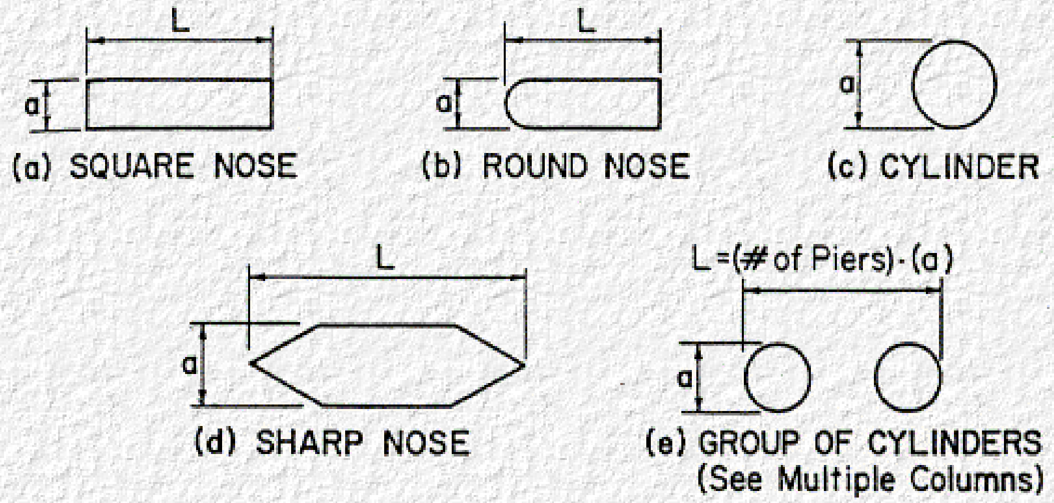

Figure 7. Common Pier Shapes

Pier Scour for Very Wide Piers. Flume studies on scour depths at wide piers in shallow flows indicate that even the CSU equation overestimates scour depth for this case.(48) Field observations of scour depths at bascule piers in shallow flows also suggest that the CSU equation overestimates scour depths. However, at the present time, there is insufficient information to estimate a decrease in scour depths given by the CSU equation for wide piers in shallow flow.

Pier Scour for Exposed Footings. Pier footings and/or pile caps may become exposed to the flow by scour. This may occur either from long-term degradation, contraction scour, or lateral shifting of the stream. Computations of local pier scour depths for footings or pile caps exposed to the flow based on footing or pile cap width appears to be too conservative. For example, calculations of scour depths for the Schoharie Creek bridge failure were closer to the measured model and prototype scour depths when pier width was used rather than footing width.(49) It appeared that the footing decreased the potential scour depth.

A model study of scour at the Acosta Bridge at Jacksonville, Florida, by Jones found that when the top of the footing was flush with the streambed, local scour was 20 percent less than for other conditions tested.(50) The other conditions were bottom of the footing at the bed surface, the top of the footing at the water surface with pile group exposed and top of footing at mid depth. In a generalized study, it was found that a footing extending upstream of the pier reduced pier scour when the top of the footing was located flush or below the bed, but scour holes became deeper and larger in proportion to the extent that the footing projected into the flow field.

Based on this study, the following recommendation was made for calculating pier scour if the footing is or may be exposed to the flow.

> "It is recommended that the pier width be used for the value of 'a' in the pier scour equations if the top of the footing (or pile cap) is at or below the streambed (after taking into account long-term degradation and contraction scour). If the pier footing extends above the streambed, make a second computation using the width of the footing for the value of "a" and the depth and average velocity in the flow zone obstructed by the footing for the 'y' and 'V' respectively in the scour equation. Use the larger of the two scour computations" (see Figure 8).

If the top of the footing or pile cap is at the long-term degradation and/or contraction scour elevation, then it is only necessary to compute the scour depth considering the pier width.

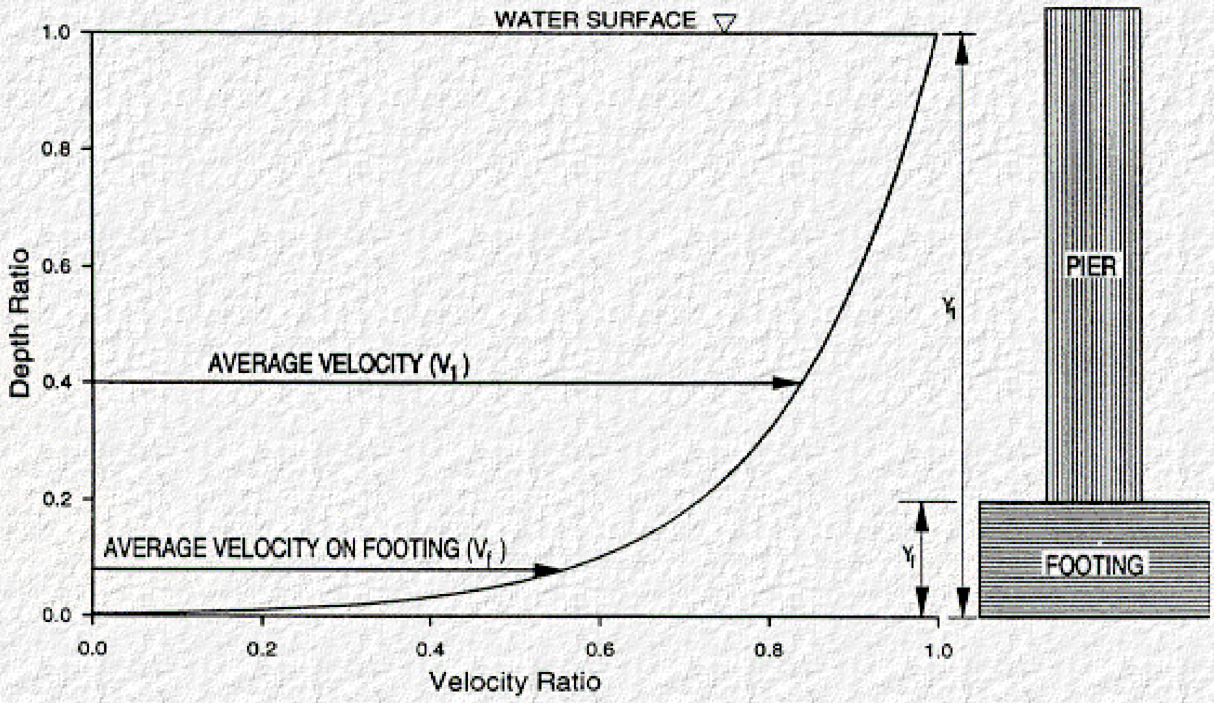

Figure 8. Definition Sketch for Velocity and Depth on Exposed Footing

Determine the average velocity of flow at the exposed footing (V_f) using the following equation:

$$\frac{V_f}{V_1} = \frac{\ln\left(10.93\,\frac{y_f}{k_s} + 1\right)}{\ln\left(10.93\,\frac{y_1}{k_s} + 1\right)} \qquad (25)$$

where:

V_f = Average velocity in the flow zone below the top of the footing, m/s
V_1 = Average velocity in the vertical of the flow approaching the pier, m/s
ln = Natural log to the base e
y_f = Distance from the bed (after degradation and contraction scour) to the top of the footing, m
k_s = Grain roughness of the bed (normally taken as the D_{84} of the bed material), m
y_1 = Depth of flow upstream of the pier, including degradation and contraction scour, m

The values of V_f and y_f would be used in Equation 21 or Equation 22 given above.

Pier Scour for Exposed Pile Groups. Experiments were conducted by Jones to determine guidelines for specifying the characteristic width of a pile group (see Figure 9) that is or may be exposed to the flow (as the result of long-term degradation and/or contraction scour) when the piles are spaced laterally as well as longitudinally in the streamflow.[50] The following was concluded:

> **"Pile groups that project above the streambed (as the result of long-term degradation and/or contraction scour) can be analyzed conservatively by representing them as a single width equal to the projected area of the piles ignoring the clear space between piles.** Good judgment needs to be used in accounting for debris because pile groups tend to collect debris that could effectively clog the clear spaces between pile and cause the pile group to act as a much larger mass."

If the pile group is exposed to the flow as the result of local scour then it is unnecessary to consider the piles in calculating pier scour.

For example, five 0.41-m cylindrical piles spaced at 1.8 m (Figure 9) would have an 'a' value of 2.05 m. This composite pier width would be used in Equation 21 to determine depth of pier scour. The correction factor K_1 in Equation 21 for the multiple piles would be 1.0 regardless of shape. If the pile group is a square as in Figure 9 or a rectangle use the dimensions as if they were a single pier and the appropriate L/a value for determining K_2 from Table 3 or calculated from Equation 24.

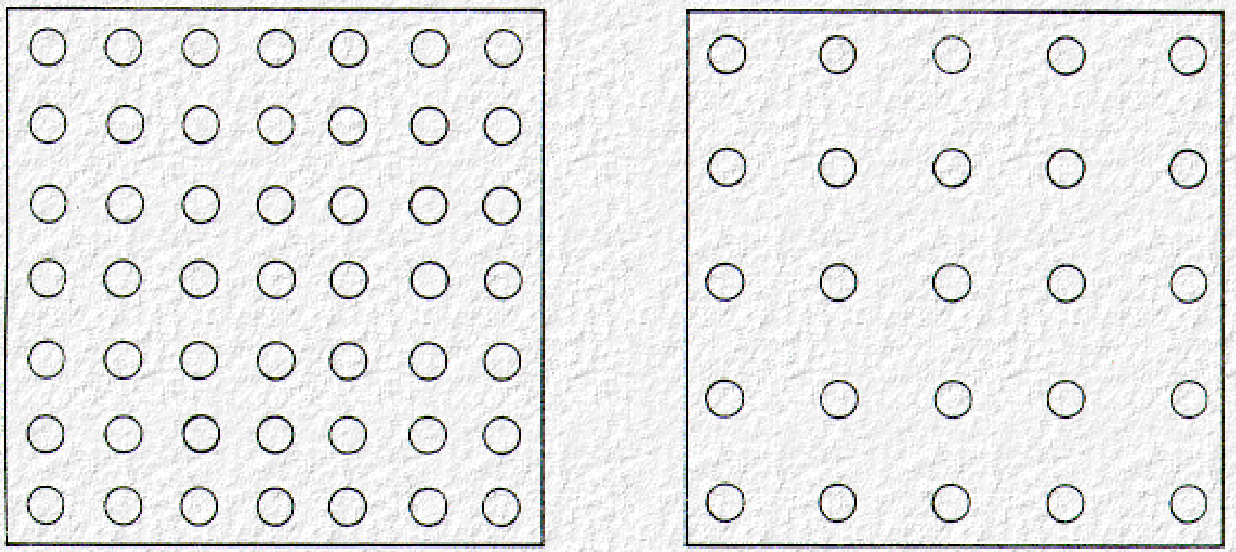

Figure 9. Pile Groups

The depth of scour for exposed pile groups will be analyzed in this manner except when addressing the effect of debris lodged between piles. If debris is a problem, it would be logical to consider the multiple columns and debris as a solid elongated pier. The appropriate L/a value and flow angle of attack would then be used to determine K_2 in Table 3.

Pile Caps Placed at the Water Surface or in the Flow. For pile caps placed at or near the water surface or in the flow (Figure 10), it is recommended that the scour analysis include computation of scour caused by the exposed pile group, computation of the pier scour caused by the pile cap and pier scour caused by the pier if the pier is partially submerged in the flow. A conservative estimate of local scour will be the largest pier scour computed from these three scenarios.

When computing the pier scour caused by the pile cap, assume that the pile cap is resting on the bed, determine V_f from Equation 25, and use the values of V_f and y_f in Equation 21. Use Equation 21 for pier shaft and exposed pile groups as recommended in the previous discussions.

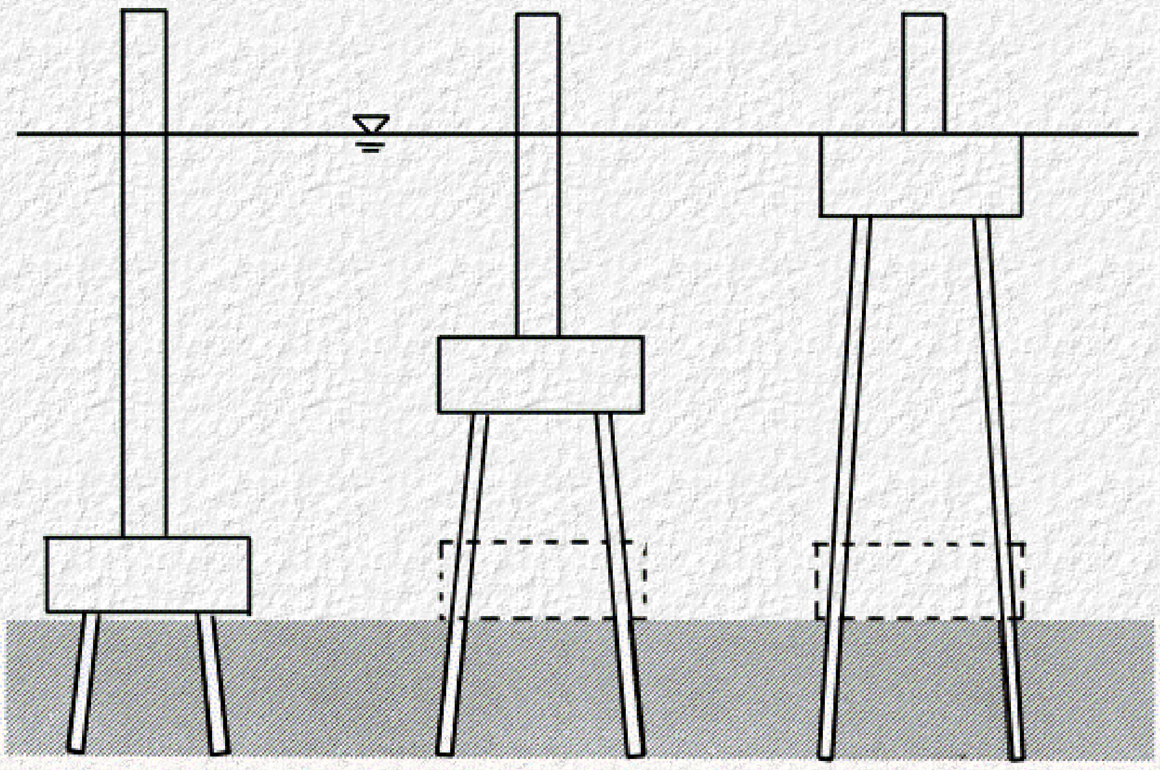

Figure 10. Pile Cap Placed at the Streambed, in the Flow, or at the Water Surface

Multiple Columns Skewed to the Flow. For multiple columns (illustrated as a group of cylinders in Figure 7) skewed to the flow, the scour depth depends on the spacing between the columns. The correction factor for angle of attack would be smaller than for a solid pier. How much smaller is not known. Raudkivi in discussing effects of alignment states "..the use of cylindrical columns would produce a shallower scour; for example, with five-diameter spacing the local scour can be limited to about 1.2 times the local scour at a single cylinder."[(27)]

In application of Equation 21 with multiple columns spaced less than 5 pier diameters apart, the pier width 'a' is the total projected width of all the columns in a single bent, normal to the flow angle of attack (see Figure 11). For example, three 2.0-m cylindrical columns spaced at 10.0 m would have an 'a' value ranging between 2.0 and 6.0 m, depending upon the flow angle of attack. **This composite pier width would be used in Equation 21 to determine depth of pier scour.** The correction factor K_1 in Equation 21 for the multiple column would be 1.0 regardless of column shape. The coefficient K_2 would also be equal to 1.0 since the effect of skew would be accounted for by the projected area of the piers normal to the flow.

If the multiple columns are spaced 5 diameter or greater apart; and debris is not a problem, limit the scour depths to a maximum of 1.2 times the local scour of a single column.

The depth of scour for a multiple column bent will be analyzed in this manner except when addressing the effect of debris lodged between columns. If debris is evaluated, it would be logical to consider the multiple columns and debris as a solid elongated pier. The appropriate L/a value and flow angle of attack would then be used to determine K_2 in Table 3 or Equation 24.

Additional laboratory studies are necessary to provide guidance on the limiting flow angles of attack for given distance between multiple columns beyond which multiple columns can be expected to function as solitary members with minimal influence from adjacent columns.

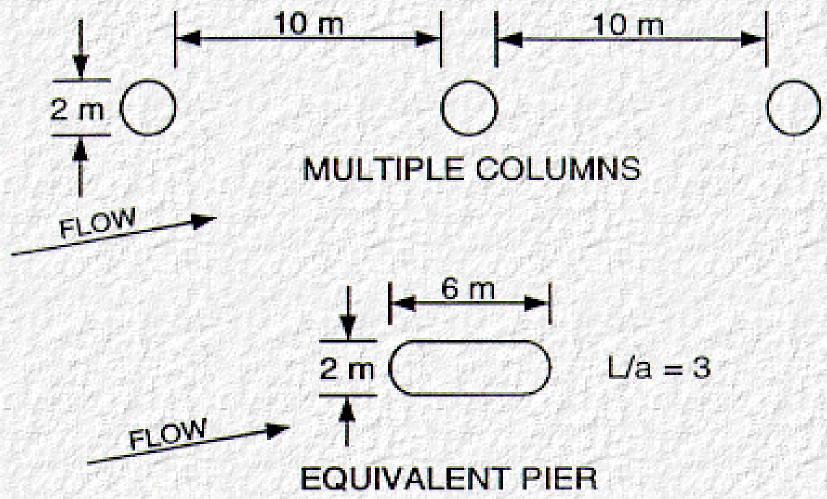

Figure 11. Multiple Columns Skewed to the Flow

Pressure Flow Scour. Pressure flow, which is also denoted as orifice flow, occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure (Figure 12). Pressure flow under the bridge results from a pile up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge (orifice flow) and flow over the bridge (weir flow).

In many cases, when a bridge is submerged, flow will also overtop adjacent approach embankments. This highway approach overtopping is also weir flow. Hence, for any overtopping situation the total weir flow can be subdivided into weir flow over the bridge and weir flow over the approach. Weir flow over approach embankments serves to reduce the discharge which must pass either under or over the bridge. In some cases, when the approach embankments are lower than the low chord of the bridge, the relief obtained from overtopping of the approach embankments will be sufficient to prevent the bridge from being submerged.

The hydraulic bridge routines of either WSPRO, HEC-2, or the new HEC River Analysis System (RAS) are suitable for determination of the amount of flow which will flow over the roadway embankment, over the bridge as weir flow, and through the bridge opening as orifice flow, provided that the top of the highway is properly included in the input data.[24,42,89] These models can be used to determine average flow depths and velocities over the road and bridge, as well as average velocities under the bridge. **It is recommended that WSPRO be used to analyze the scour problem when the bridge is overtopped with or without overtopping of the approach roadway.**

With pressure flow, the local scour depths at a pier or abutment are larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subjected to pressure flow results from the flow being directed downward towards the bed by the superstructure (vertical contraction of the flow) and by increasing the intensity of the horseshoe vortex. The vertical contraction of the flow can be a more significant cause of the increased scour depth. However, in many cases, when a bridge becomes submerged, the average velocity under the bridge is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow, and a reduction of the discharge which must pass under the bridge due to weir flow over the bridge and/or approach embankments. As a consequence of this, **increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lower velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to overtopping of the bridge and approach embankments.**

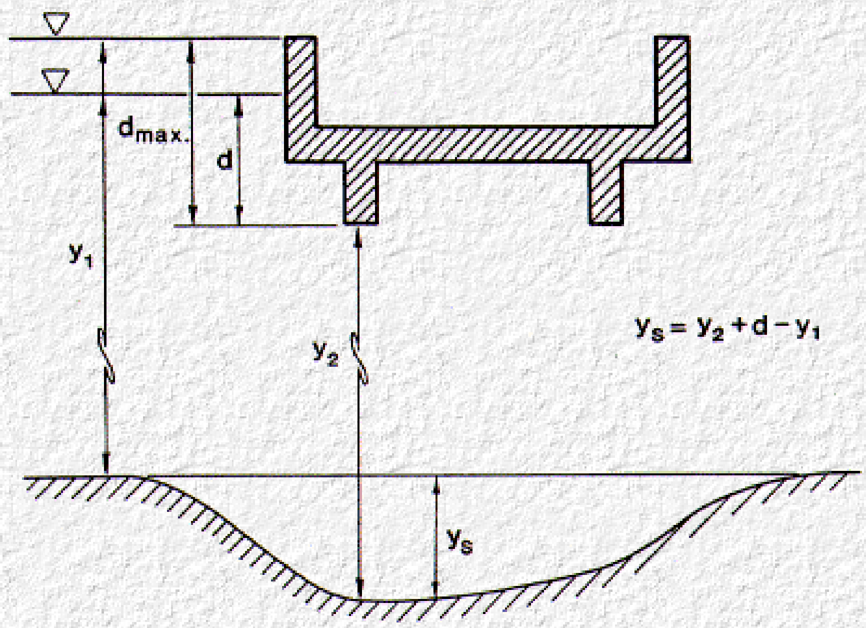

Figure 12. Definition Sketch of Vertical Contraction Scour Resulting From Pressure Flow (see Appendix B)

Limited studies of pressure flow scour have been made in flumes at Colorado State University and FHWA's Turner Fairbank Highway Research Center which indicate that pier scour can be increased 200 to 300 percent by pressure flow.[51,52,53] Both studies were for clear-water scour (no transport of bed material upstream of the bridge). FHWA's Turner Fairbank study indicates that local pier scour with pressure flow is a combination of a deck scour component and a local pier scour component. The deck scour component is a form of vertical contraction scour. As a result, the components are additive. In addition, the FHWA study observed that the pressure-flow pier scour component was approximately the same as the free-surface pier scour measurements for the same approach flow condition.

In Appendix B a revision of the Jones et al. (1993) paper is given, which presents interim procedures for computing the deck scour component for pressure-flow pier scour under clear-water conditions.[54] This deck scour component would be added to the local pier scour component to obtain the total clear-water, pressure-flow pier scour depth. The local pier scour component is calculated using Equation 21 and the approach flow depth and velocity.

The interim procedures can be used to obtain an estimate of live-bed, pressure-flow pier scour depth. However, the calculated scour depths would be conservative. That is, the calculated depths would be deeper than those to be expected with live-bed, pressure-flow pier scour.

Scour from Debris on Piers. Debris lodged on a pier also increases local scour at a pier. The debris may increase pier width and deflect a component of flow downward. This increases the transport of sediment out of the scour hole. When floating debris is lodged on the pier, the scour depth can be estimated by assuming that the pier width is larger than the actual width. The problem is in determining the increase in pier width to use in the pier scour equation. Furthermore, at large depths, the effect of the debris on scour depth should diminish.

As with estimating local scour depths with pressure flow, only limited research has been done on local scour with debris. Melville and Dongol have conducted a limited quantitative study of the effect of debris on local pier scour and have made some recommendations which support the approach suggested above.[55] However, additional laboratory studies will be necessary to better define the influence of debris on local scour.

An interim procedure for estimating the effect of debris on local scour at piers is presented in Appendix G.

Width of Scour Holes. The topwidth of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the following equation:[56]

$$W = y_s (K + \cot\theta) \qquad (27)$$

where:

W = Topwidth of the scour hole from each side of the pier or footing, m
y_s = Scour depth, m
K = Bottom width of the scour hole as a fraction of scour depth
θ = Angle of repose of the bed material ranging from about 30° to 44°

The angle of response of cohesiveness material in air ranges from about 30° to 44°. Therefore, if the bottom width of the scour hole is equal to the depth of scour y_s (K = 1), the topwidth in cohesionless sand would vary from 2.07 to 2.80 y_s. At the other extreme, if K = 0, the topwidth would vary from 1.07 to 1.8 y_s. Thus, the topwidth could range from 1.0 to 2.8 y_s and will depend on the bottom width of the scour hole and composition of the bed material. In general, the deeper the scour hole, the smaller the bottom width. In water, the angle of repose of cohesionless material is less than the values given for air; therefore, a topwidth of 2.0 y_s is suggested for practical applications (Figure 13).

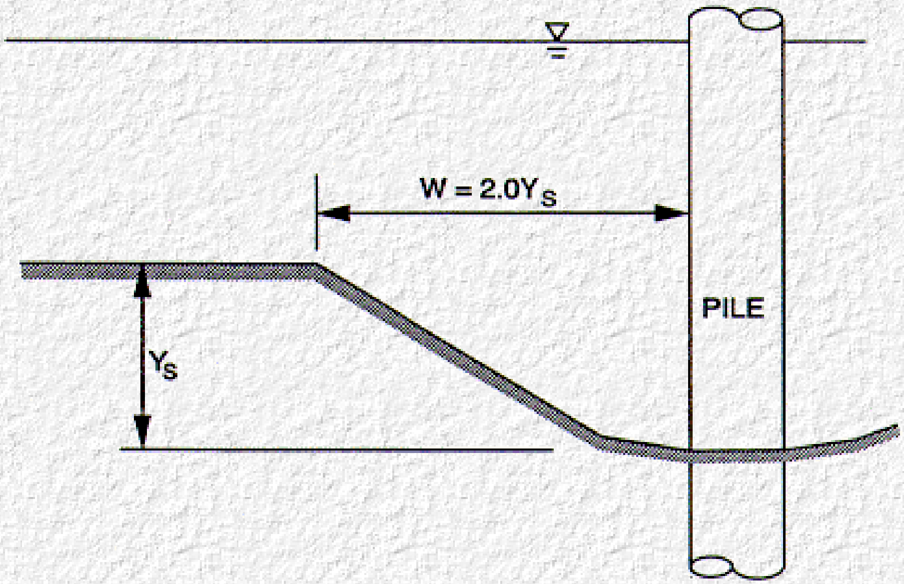

Figure 13. Topwidth of Scour Hole

4.3.6 Step 6: Local Scour at Abutments

General. Local scour occurs at abutments when the abutment obstructs the flow. The obstruction of the flow forms a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment, and a vertical wake vortex at the downstream end of the abutment. The vortex at the toe of the abutment is very similar to the horseshoe vortex that forms at piers, and the vortex that forms at the downstream end is similar to the wake vortex that forms downstream of a pier, or that forms downstream of any flow separations. Research has been conducted to determine the depth and location of the scour hole that develops for the horizontal (so called horseshoe) vortex that occurs at the upstream end of the abutment, and numerous abutment scour equations have been developed to predict this scour depth. However, abutment failures and erosion of the approach have occurred from the action of the downstream wake vortex. Although research is lacking on the scour caused by this downstream wake vortex, the abutment can be protected by using riprap to protect the downstream toe and approach. Some State DOTs protect the downstream toe and embankment by constructing a 15-m guide bank (see HEC-20).[12] Chapter 7 presents a procedure for designing rock riprap to protect bridge abutments from scour.

Equations for predicting abutment scour depths such as Liu et al., Laursen, Froehlich, and Melville are based entirely on laboratory data.[57,46,58,59] The problem is that little field data on abutment scour exist. Liu et al.'s equations were developed by dimensional analysis of the variables with a best-fit line drawn through the laboratory data.[57] Laursen's equations are based on inductive reasoning of the change in transport relations due to the acceleration of the flow caused by the abutment.[40] Froehlich's equations were derived from dimensional analysis and regression analysis of the available laboratory data.[51] Melville's equations were derived from dimensional analysis and development of relations between dimensionless parameters using best-fit lines through laboratory data.[59]

All equations in the literature were developed using the abutment and roadway approach length as one of the variables and result in excessively conservative estimates of scour depth. Richardson and Richardson pointed this out in a discussion of Melville's (1992) paper,[60,59]

> "The reason the equations in the literature predict excessively conservative abutment scour depths for the field situation is that, in the laboratory flume, the discharge intercepted by the abutment is directly related to the abutment length; whereas, in the field, this is rarely the case."

Figure 14 illustrates the difference. Thus, using the abutment length in the equations instead of the discharge returning to the main channel at the abutment results in a spurious correlation between abutment lengths and scour depth at the abutment.

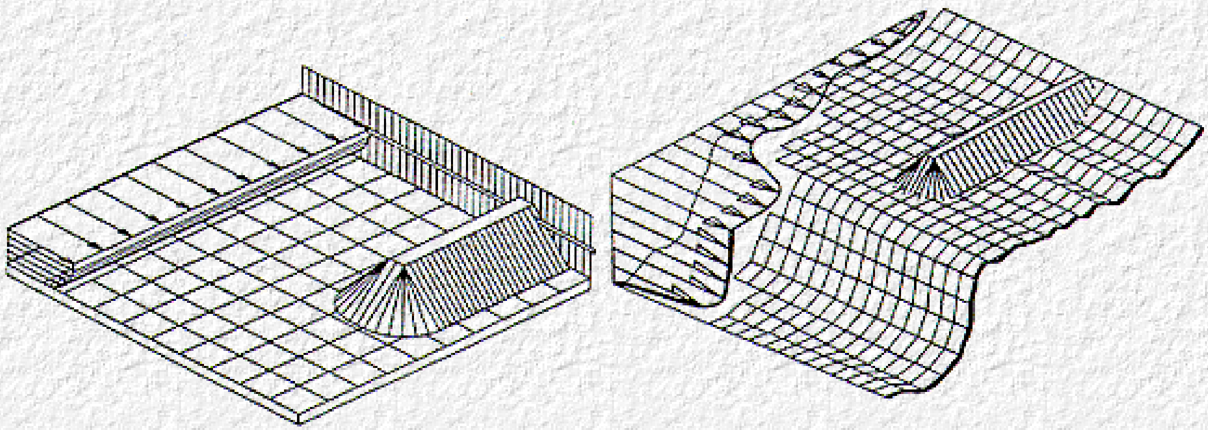

Figure 14. Comparison of Laboratory Flow Characteristics to Field Flow Conditions

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length in the field case. Richardson and Richardson noted that abutment scour depth depends on abutment shape, discharge in the main channel at the abutment, discharge intercepted by the abutment and returned to the main channel at the abutment, sediment characteristics, cross-sectional shape of the main channel at the abutment (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), alignment, etc.[60] In addition, field conditions may have tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment. **Most of the laboratory research to date has failed to replicate these field conditions**.

Therefore, engineering judgment is required in designing foundations for abutments. In many cases, foundations can be designed with shallower depths than predicted by the equations when they are protected with rock riprap as recommended in Chapter 7 and/or with a guide bank placed upstream of the abutment. Cost will be the deciding factor. A method to determine the length of a guide bank is given in HEC-20.[12]

In the following sections, two equations are presented for use in estimating scour depths as a guide in designing abutment foundations. As stated above, these equations generally give excessively conservative estimates of scour depths.

Abutment Site Conditions. Abutments can be set back from the natural streambank or project into the channel. Scour at abutments can be live-bed or clear-water scour. Finally, there can be varying amounts of overbank flow intercepted by the approaches to the bridge and returned to the stream at the abutment. More severe abutment scour will occur when the majority of overbank flow returns to the bridge opening directly upstream of the bridge crossing. Less severe abutment scour will occur when overbank flows gradually return to the main channel upstream of the bridge crossing.

Abutment Shape. There are three general shapes for abutments:

1. spill-through abutments,
2. vertical walls without wing walls, and
3. vertical-wall abutments with wing walls (Figure 15).

These shapes can be set at varying angles to the flow. Depth of scour is approximately double for vertical-wall abutments as compared with spill-through abutments.

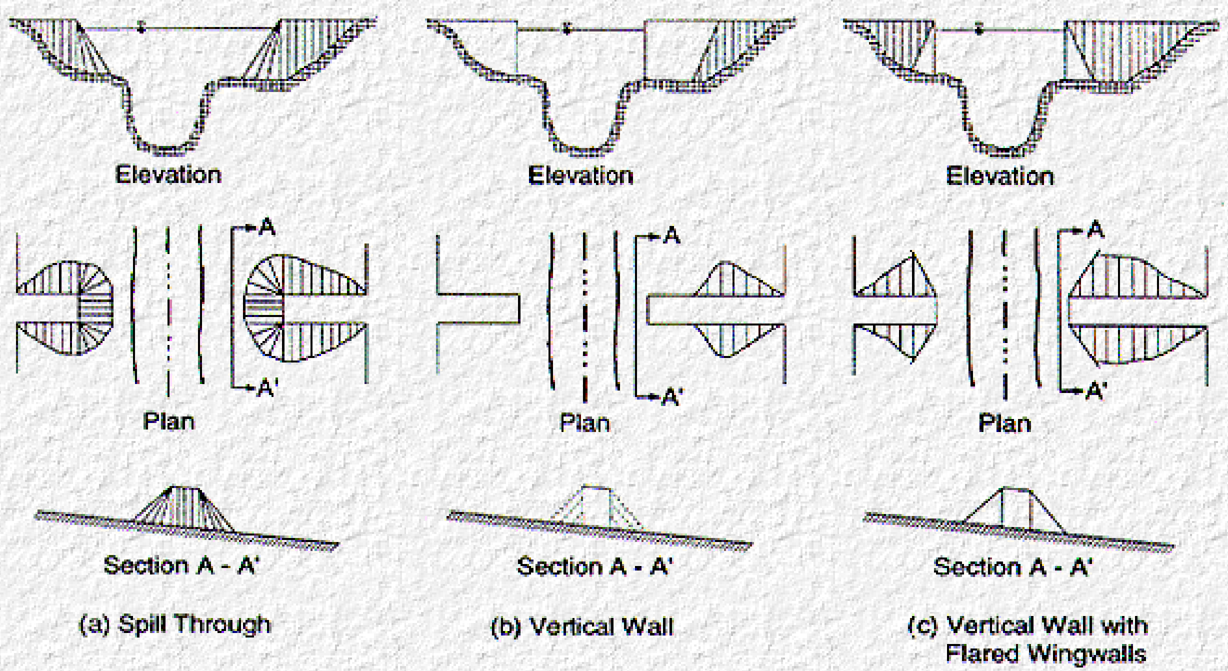

Figure 15. Abutment Shape

Design for Scour at Abutments. The potential for lateral channel migration, long-term degradation and contraction scour should be considered in setting abutment foundation depths near the main channel. It is recommended that the abutment scour equations be used to develop insight as to the scour potential of an abutment. Then, the abutment may be designed to resist the computed scour or as an alternative, riprap and guide banks can be used to protect the abutment from scour and erosion. Normally, protection is provided using rock riprap with the guidance from Chapter 7 and/or guide banks designed as given in HEC-20.[12] **Engineering judgment is required in setting foundation depths for abutments.**

Live-Bed Scour at Abutments. As a check on the potential depth of scour to aid in the design of the foundation and placement of rock riprap or guide banks, Froehlich's live-bed scour equation or an equation from HIRE can be used.(58) Froehlich analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following equation:

$$\frac{y_s}{y_a} = 2.27\ K_1\ K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \tag{28}$$

where:

K_1 = Coefficient for abutment shape (seeTable 6)
K_2 = Coefficient for angle of embankment to flow
K_2 = $(\theta/90)^{0.13}$ (see Figure 16 for definition of θ)
$\theta<90^0$ if embankment points downstream
$\theta>90^0$ if embankment points upstream
L' = Length of abutment (embankment) projected normal to flow, m
A_e = Flow area of the approach cross section obstructed by the embankment, m^2
Fr = Froude Number of approach flow upstream of the abutment
= $V_e/(gy_a)^{1/2}$
Q_e/A_e, m/s
V_e = Flow obstructed by the abutment and approach embankment, m^3/s
Q_e = Average depth of flow on the floodplain, m
y_a = Scour depth, m
y_s =

It should be noted that Equation 28 is not consistent with the fact that as L´ tends to 0, y_s also tends to 0. The 1 was added to the equation so as to envelope 98 percent of the data.

Table 6. Abutment Shape Coefficients

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

An equation based on field data of scour at the end of spurs in the Mississippi River (obtained by the USACOE) can also be used for estimating abutment scour.(13) This field situation closely resembles the laboratory experiments for abutment scour in that the discharge intercepted by the spurs was a function of the spur length. The equation, referred to herein as the HIRE equation, is applicable when the ratio of projected abutment length (L´) to the flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_s) at an abutment where conditions are similar to the field conditions from which the equation was derived:

$$\frac{y_s}{y_1} = 4 Fr^{0.33} \frac{K_1}{0.55} \tag{29}$$

where:

y_s = Scour depth, m
y_1 = Depth of flow at the abutment on the overbank or in the main channel, m
Fr = Froude Number based on the velocity and depth adjacent to and upstream of the abutment
K_1 = Abutment shape coefficient (from Table 6)

To correct Equation 29 for abutments skewed to the stream, use Figure 16.(13)

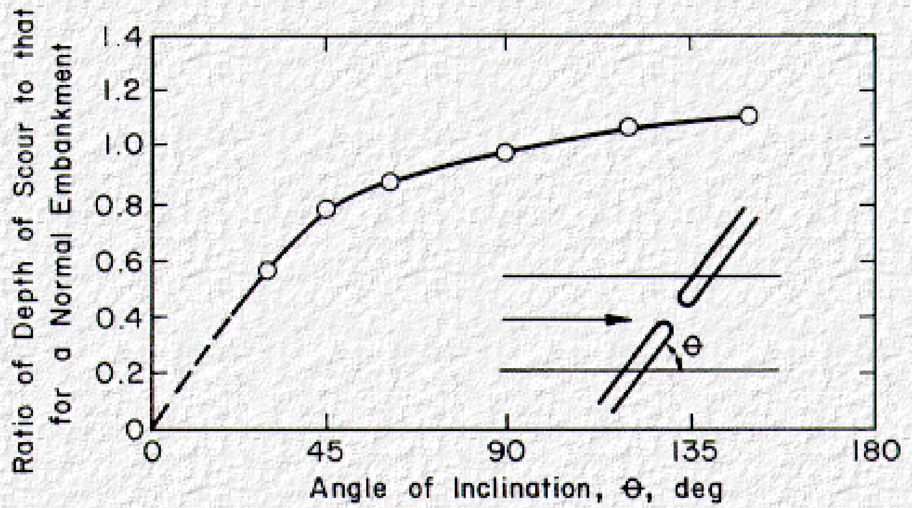

Figure 16. Adjustment of Abutment Scour Estimate for Skew

Clear-Water Scour at an Abutment. Use Equation 28 or Equation 29 for live-bed scour because clear-water scour equations potentially decrease scour at abutments due to the presence of coarser material. This decrease is unsubstantiated by field data.

4.3.7 Step 7: Plot Total Scour Depths and Evaluate Design

Plot the Total Scour Depths. On the cross section of the stream channel and floodplain at the bridge crossing, plot the estimate of long-term bed elevation change, contraction scour, and local scour at the piers and abutments. Use a distorted scale so that the scour determinations will be easy to evaluate. Make a sketch of any planform changes (lateral stream channel movement due to meander migration, etc.) that might be reasonably expected to occur.

1. Long-term elevation changes may be either aggradation or degradation. However, only degradation is considered in scour computations.
2. Contraction scour is then plotted from and below the long-term degradation line.
3. Local scour is then plotted from and below the contraction scour line.
4. Plot not only the depth of scour at each pier and abutment, but also the scour hole width. Use 2.0 y_s to estimate scour hole width on each side of the pier.

Evaluate the Total Scour Depths.

1. Evaluate whether the computed scour depths are reasonable and consistent with the design engineer's previous experience, and engineering judgment. If not, modify the depths to reflect sound engineering judgment.
2. Evaluate whether the local scour holes from the piers or abutments overlap between spans. If so, local scour depths can be larger though indeterminate. For new or replacement bridges, the length of the bridge opening should be reevaluated and the opening increased or the number of piers decreased as necessary to avoid overlapping scour holes.
3. Evaluate other factors such as lateral movement of the stream, streamflow hydrograph, velocity and discharge distribution, movement of the thalweg, shifting of the flow direction, channel changes, type of stream, or other factors.
4. Evaluate whether the calculated scour depths appear too deep for the conditions in the field, relative to the laboratory conditions. **Abutment scour equations are for the worst-case conditions.** Rock riprap or a guide bank could be a more cost-effective solution than designing the abutment to resist the computed abutment scour depths.
5. Evaluate cost, safety, etc. Also, account for ice and/or debris effects.
6. In the design of bridge foundations, the bottom foundation elevation(s) should be at or below the total scour elevation(s) as discussed in Chapter 3.

Reevaluate the Bridge Design. Reevaluate the bridge design on the basis of the foregoing scour computations and evaluation. Revise the design as necessary. This evaluation should consider the following questions:

1. Is the waterway area large enough (i.e., is contraction scour too large)?
2. Are the piers too close to each other or to the abutments (i.e., do the scour holes overlap)? Estimate the topwidth of a scour hole on each side of a pier at 2.0 times the depth of scour. If scour holes overlap, local scour can be deeper.
3. Is there a need for relief bridges? Should they or the main bridge be larger?
4. Are bridge abutments properly aligned with the flow and located properly in regard to the stream channel and floodplain?
5. Is the bridge crossing of the stream and floodplain in a desirable location? If the location presents problems:
    . Can it be changed?
    b. Can river training works, guide banks, or relief bridges serve to provide for an acceptable flow pattern at the bridge?
6. Is the hydraulic study adequate to provide the necessary information for foundation design?
    . Are flow patterns complex?
    b. Should a two-dimensional, water-surface profile model be used for analysis?
    c. Is the foundation design safe and cost-effective?
    d. Is a physical model study needed/warranted?

Go to Chapter 4, Part II

Chapter 4 : HEC 18

Estimating Scour at Bridges
Part II

Go to Chapter 4, Part III

4.4 Scour Example Problem

4.4.1 General Description of Problem

This example problem is taken from a paper by Arneson et al.[62] FHWA's WSPRO computer program was used to obtain the hydraulic variable. The program uses 20 stream tubes to give a quasi two-dimensional analysis. The stream tubes provide the velocity distribution across the flow and the program has excellent bridge routines.

A 198.12-m long bridge (Figure 17) is to be constructed over a channel with spill-through abutments (slope of 1V:2H). The left abutment is set approximately 60.5 m back from the channel bank. The right abutment is set at the channel bank. The bridge deck is set at elevation 6.71 m and has a girder depth of 1.22 m. Six round-nose piers are evenly spaced in the bridge opening. The piers are 1.52 m thick, 12.19 m long, and are aligned with the flow. The 100-year design discharge is 849.51 m^3/s. The 500-year flow of 1444.16 m^3/s was estimated by multiplying the Q_{100} by 1.7 since no hydrologic records were available to predict the 500-year flow.

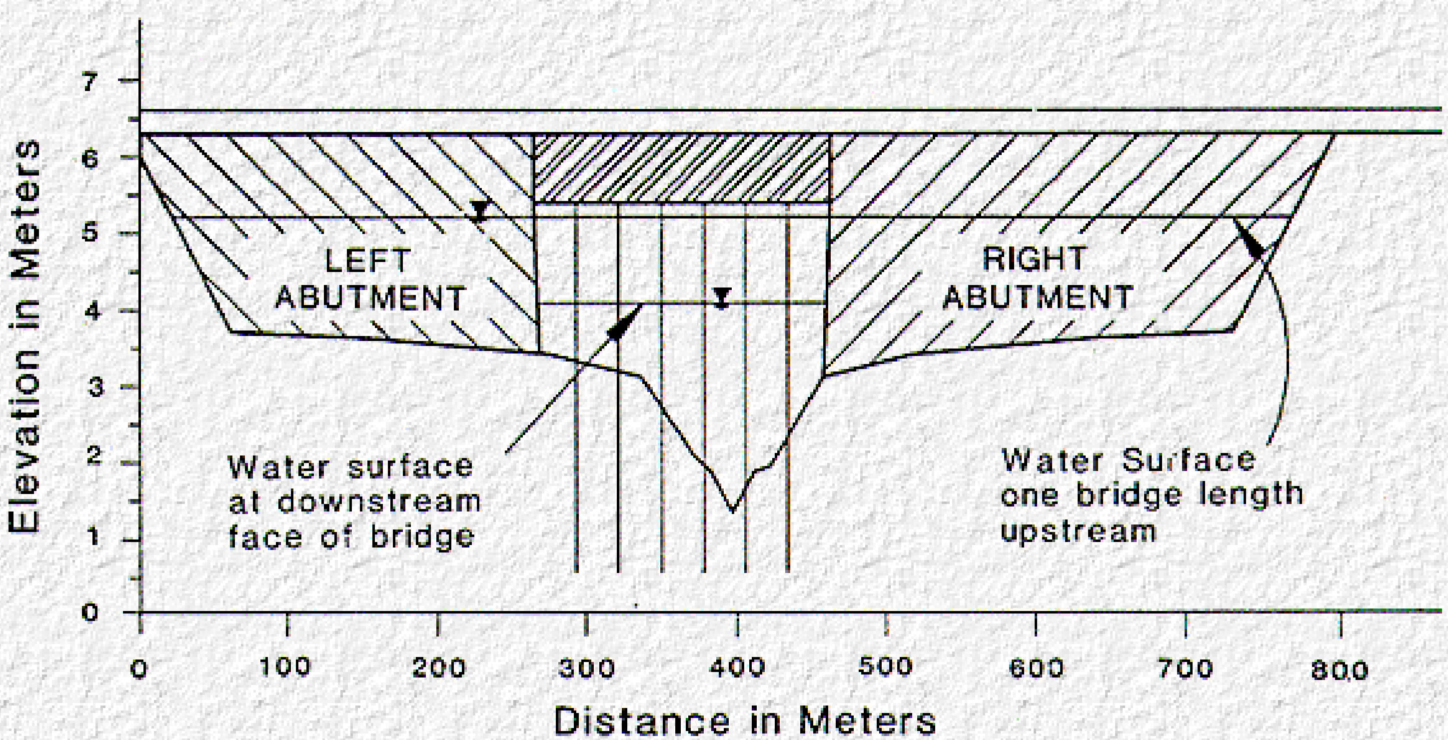

Figure 17. Cross Section of Proposed Bridge

4.4.2 Step 1: Determine Scour Analysis Variables

From Level 1 and Level 2 analysis: a site investigation of the crossing was conducted to identify potential stream stability problems at this crossing. Evaluation of the site indicates that the river has a relatively wide floodplain. The floodplain is well vegetated with grass and trees; however, the presence of remnant channels indicates that there is a potential for lateral shifting of the channel.

The bridge crossing is located on a relatively straight reach of channel. The channel geometry is relatively the same for approximately 300 m up- and downstream of the bridge crossing. The D_{50} of the bed material and overbank material is approximately 0.002 m (2 mm). The maximum grain size of the bed material is approximately 0.008 m (8 mm). The specific gravity of the bed material was determined to be equal to 2.65.

The river and crossing are located in a rural area with the primary land use consisting of agriculture and forest.

Review of bridge inspection reports for bridges located upstream and downstream of the proposed crossing indicates no long-term aggradation or degradation in this reach. At the bridge site, bedrock is approximately 46 m below the channel bed.

Since this is a sand-bed channel, no armoring potential is expected. Furthermore, the bed for this channel at low flow consists of dunes which are approximately 0.3 to 0.5 m high. At higher flows, above the Q_5, the bed will be either plane bed or antidunes.

The left and right banks are relatively well vegetated and stable; however, there are isolated portions of the bank which appear to have been undercut and are eroding. Brush and trees grow to the edge of the banks. Banks will require riprap protection if disturbed. Riprap will be required upstream of the bridge and extend downstream of the bridge.

Hydraulic Characteristics. Hydraulic characteristics at the bridge were determined using WSPRO.[24] Three cross sections were used for this analysis and are denoted as "EXIT" for the section downstream of the bridge, "FULLV" for the full-valley section at the bridge, and "APPR" for the approach section located one bridge length upstream of the bridge. The bridge geometry was superimposed on the full-valley section and is denoted "BRDG." Values used for this example problem are based on the output from the WSPRO model which is presented in Appendix C. Specific values for scour analysis variables are given for each computation separately and cross referenced to the line numbers of the WSPRO output.

The HP2 option was used to provide hydraulic characteristics at both the bridge and approach sections. This WSPRO option subdivides the cross section into 20 equal conveyance tubes. Figure 18 and Figure 19 illustrate the location of these conveyance tubes for the approach and bridge cross section, respectively. Figure 20 illustrates the average velocities in each conveyance tube and the contraction of the flow from the approach section through the bridge. Figure 20 also identifies the equal conveyance tubes of the approach section which are cut off by the abutments.

Hydraulic variables for performing the various scour computations were determined from the WSPRO output (see Appendix C) and from Figure 18, Figure 19, and Figure 20. These variables which will be used to compute contraction scour and local scour are presented in Table 7, Table 8, Table 9, Table 10, Table 11, and Table 12.

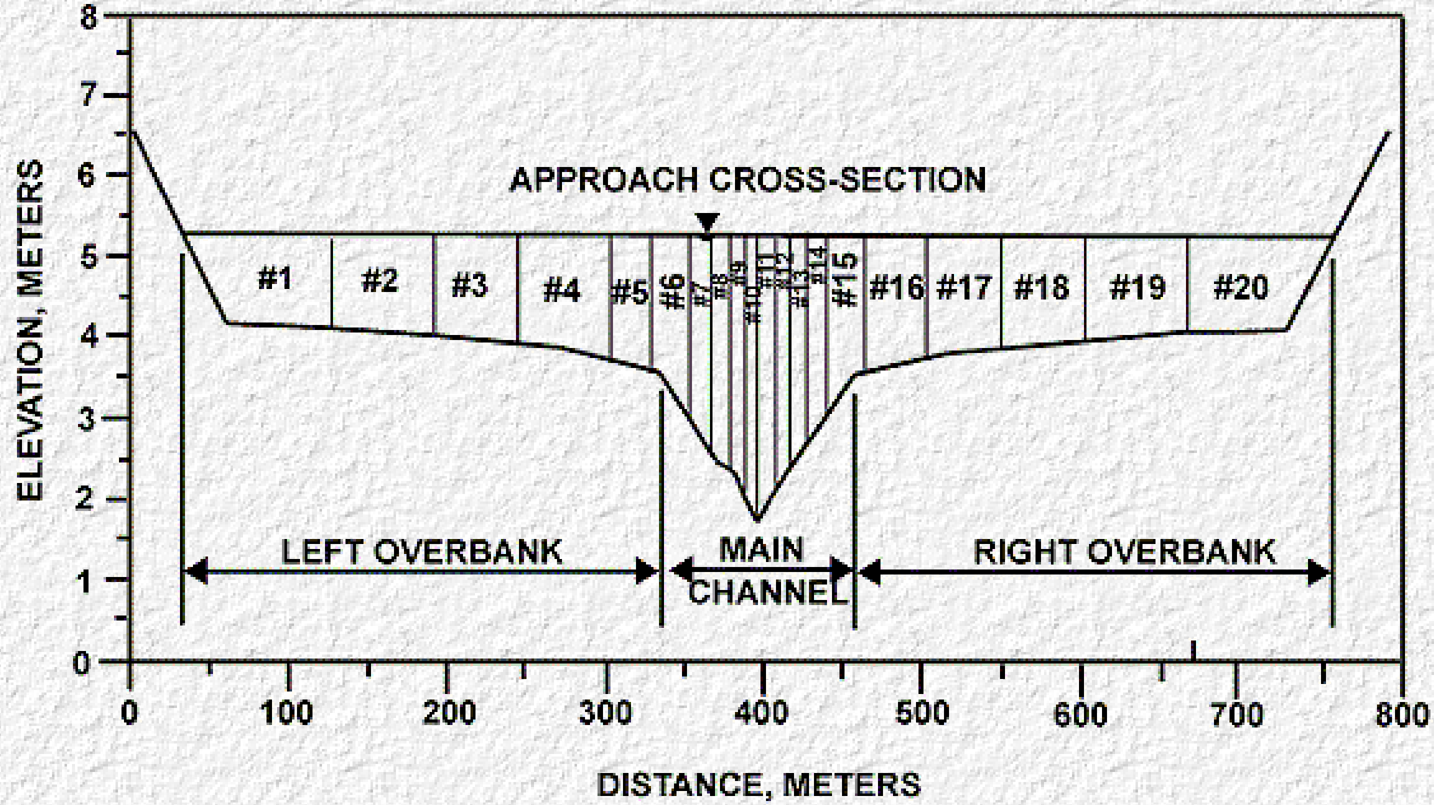

Figure 18. Equal Conveyance Tubes of Approach Section

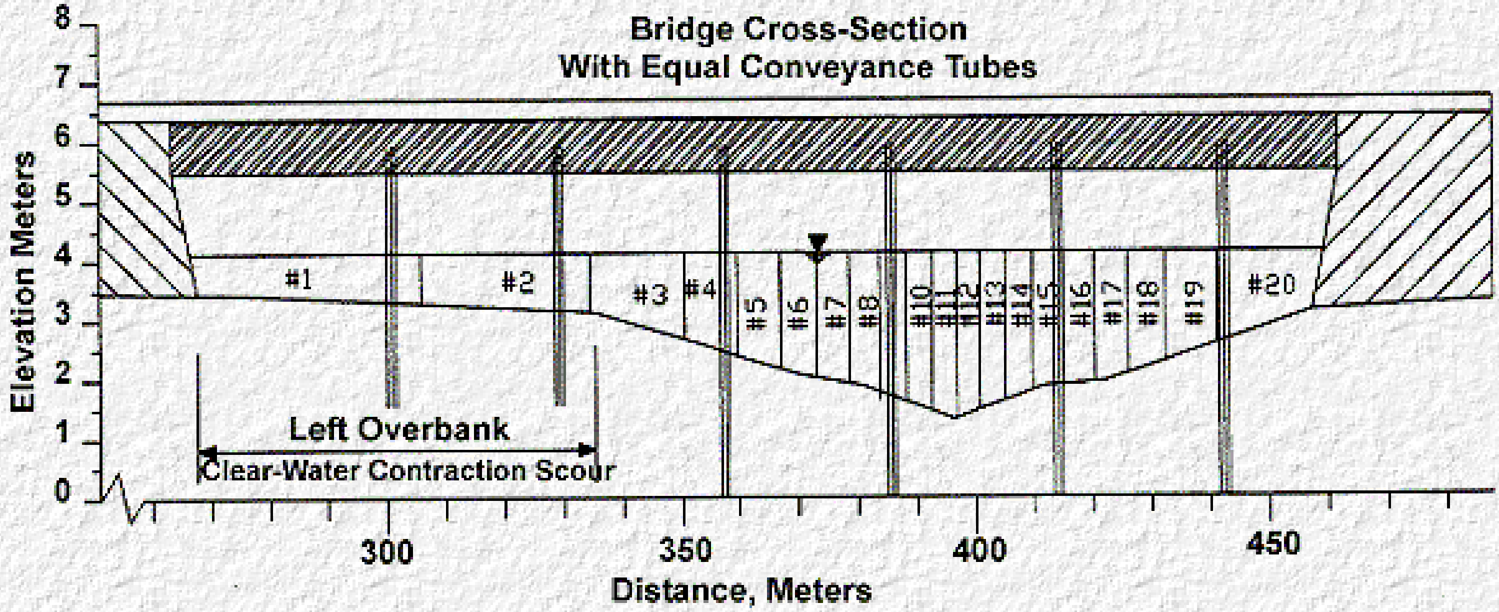

Figure 19. Equal Conveyance Tubes of Bridge Section

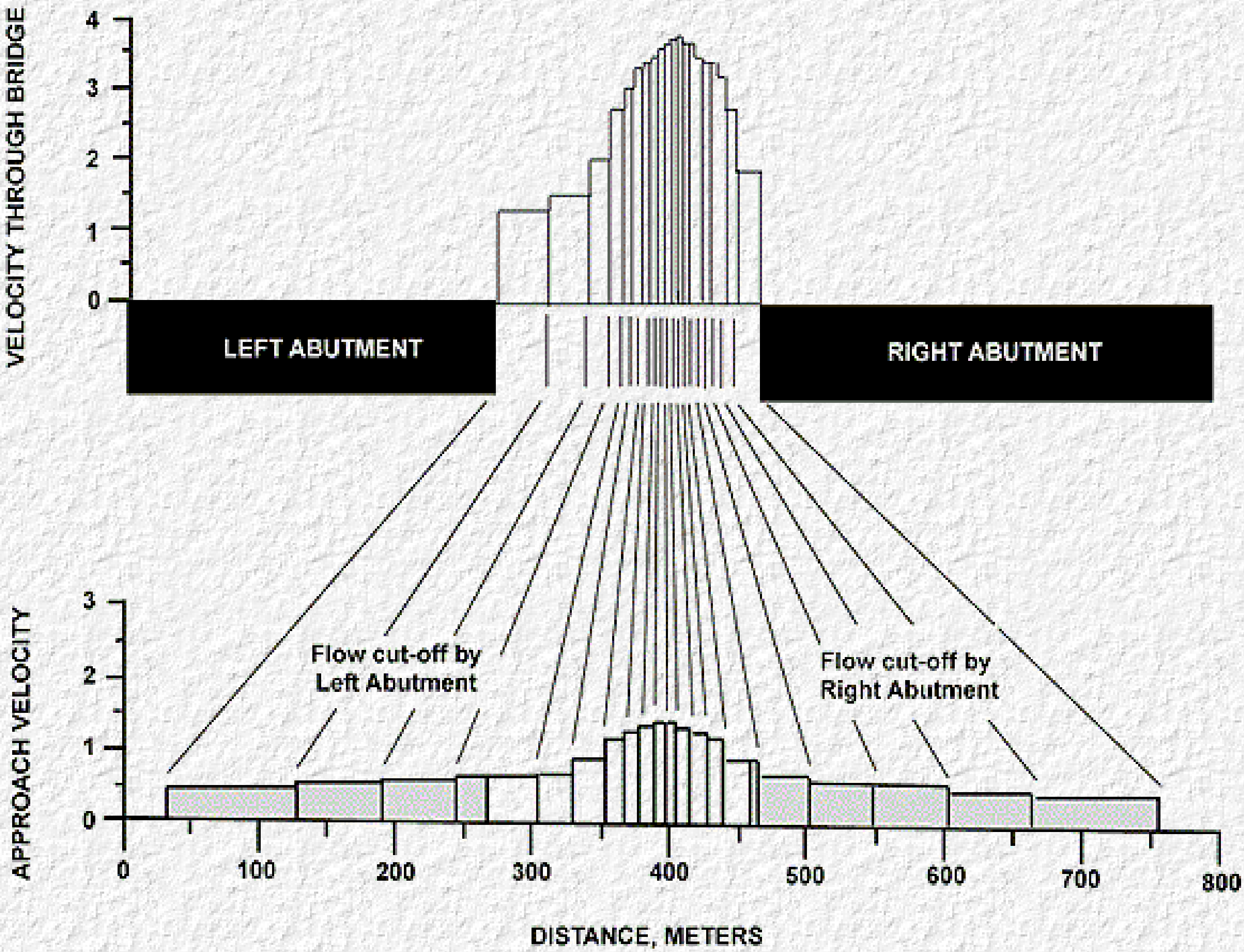

Figure 20. Plan View of Equal Conveyance Tubes Showing Velocity Distribution at Approach and Bridge Sections

Table 7. Hydraulic Variables from WSPRO for Estimation of Live-bed Contraction Scour

		Remarks
Q (m^3/s)	849.51	Total discharge, line 8 of WSPRO input or Line 26 of WSPRO output.
K_1 (Approach)	19 000	Conveyance of main channel of approach. Line 378 of WSPRO output, SA#2.
K_{total} (Approach)	39 150	Total conveyance of approach section. Line 380 of WSPRO output.
W_1 or TOPW (Approach) (m)	121.9	Top width of flow (TOPW). Assumed to represent active live bed width of approach. Line 378 of WSPRO output, SA#2.
A_c (Approach) (m^2)	320	Area of main channel approach section. Line 378, SA#2.
WETP (Approach) (m)	122.0	Wetted perimeter of main channel approach section. Line 378 of WSPRO output, SA#2.
K_c (Bridge)	11 330	Conveyance of main channel through bridge. Line 334 of WSPRO output, SA#2.
K_{total} (Bridge)	12 540	Total conveyance through bridge. Line 335 of WSPRO output.

A_c (Bridge) (m^2)	236	Area of the main channel, bridge section. Line 334 of WSPRO output, SA #2.
W_c (Bridge) (m)	122	Channel width at the bridge. Difference between subarea break-points defining banks at bridge, line 109 of WSPRO output.
W_2 (Bridge) (m)	115.9	Channel width at bridge, less 4 channel pier widths (6.08 m).
S_f (m/m)	0.002	Average unconstricted energy slope (SF). Line 260, or 266 of WSPRO output.

Table 8. Hydraulic Variables from WSPRO for Estimation of Clear-water Contraction Scour on Left Overbank

		Remarks
Q (m^3/s)	849.51	Total discharge, (see Table 7).
Q_{chan} (Bridge) (m^3/s)	767.54	Flow in main channel at bridge. Determined in live-bed computation of step 5A.
Q_2 (Bridge) (m^3/s)	81.97	Flow in left overbank through bridge. Determined by subtracting Q_{chan} (listed above) from total discharge through bridge.
D_m (Bridge Overbank) (m)	0.0025	Grain size of left overbank area. $D_m = 1.25\ D_{50}$.
$W_{setback}$ (Bridge)(m)	68.8	Top width of left overbank area (SA #1) at bridge. Line 333, of WSPRO output.
$W_{contracted}$ (Bridge) (m)	65.8	Set back width less two pier widths (3.04 m).
A_{left} (Bridge) (m^2)	57	Area of left overbank at the bridge. Line 333 of WSPRO output, SA #1.

Table 9. Hydraulic Variables from WSPRO for Estimation of Pier Scour (Conveyance Table Number 12).

		Remarks
V_1 (m/s)	3.73	Velocity in conveyance tube #12. Line 315 of WSPRO output.
Y_1 (m)	2.84	Mean depth of tube #12. Line 316 of WSPRO output.

Table 10. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using Froehlich's Equation for Left Abutment[57]

		Remarks
Q (m^3/s)	849.51	Total discharge (see Table 7).
q_{tube} (m^3/s)	42.48	Discharge per equal conveyance tube, defined as total discharge divided by 20.
#Tubes	3.5	Number of approach section conveyance tubes which are obstructed by left abutment. Determined by superimposing abutment geometry onto the approach section (Figure 20).
Q_e (m^3/s)	148.68	Flow in left overbank obstructed by left abutment. Determined by multiplying # Tubes and q_{tube}.
A_e (left abut.) (m^2)	264.65	Area of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 348 of WSPRO output.

L' (m)	232.80	Length of abutment projected into flow, determined by adding top widths of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 347 of WSPRO output.

Table 11. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Left Abutment(13)

		Remarks
V_{tube} (m/s) (Bridge x-Section)	1.29	Mean velocity of conveyance tube #1, adjacent to left abutment. Line 305 of WSPRO output.
y_1 (m) (Bridge x-Section)	0.83	Average depth of conveyance tube #1. Line 306 of WSPRO output.

Table 12. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Right Abutment(13)

		Remarks
V_{tube} (m/s)	2.19	Mean velocity of conveyance tube 20, adjacent to right abutment. Line 320 of WSPRO output.
y_1 (m)	1.22	Average depth of conveyance tube 20. Line 321 of WSPRO output.

Contraction scour will occur both in the main channel and on the left overbank of the bridge opening. For the main channel, contraction scour could be either clear-water or live-bed depending on the magnitude of the channel velocity and the critical velocity for sediment movement. A computation will be performed to determine the sediment transport characteristics of the main channel and the appropriate contraction scour equation.

In the overbank area adjacent to the left abutment, clear-water scour will occur. This is because the overbank areas upstream of the bridge are vegetated, and because the velocities in these areas will be low. Thus, returning overbank flow which will pass under the bridge adjacent to the left abutment will not be transporting significant amounts of material to replenish the scour on the left overbank adjacent to the left abutment.

Because of this, two computations for contraction scour will be required. The first computation, which will be illustrated in step 4-A will determine the magnitude of the contraction scour in the main channel. The second computation, which is illustrated in step 4-B will utilize the clear-water equation for the left overbank area. Hydraulic data for these two computations are presented in Table 7 and Table 8 for the channel and left overbank contraction scour computations respectively.

Table 9 lists the hydraulic variables which will be used to estimate the local scour at the piers (step 5). These hydraulic variables were determined from a plot of the velocity distribution derived from the WSPRO output (Figure 21). For this example the highest velocities and flow depths in the bridge cross section will be used (at conveyance tube number 12). Only one pier scour computation will be computed because the possibility of thalweg shifting and lateral migration will require that all of the piers be set assuming that any pier could be subjected to the maximum scour producing variables.

Local scour at the left abutment will be illustrated in step 6-A using the Froehlich and HIRE equations.[58,13] Scour variables derived from the WSPRO output for these two computations are presented in Table 10 and Table 11 for the Froehlich and HIRE equation respectively. Local scour at the right abutment will be computed in Step 6-B using the HIRE equation, and the hydraulic variables listed in Table 12.

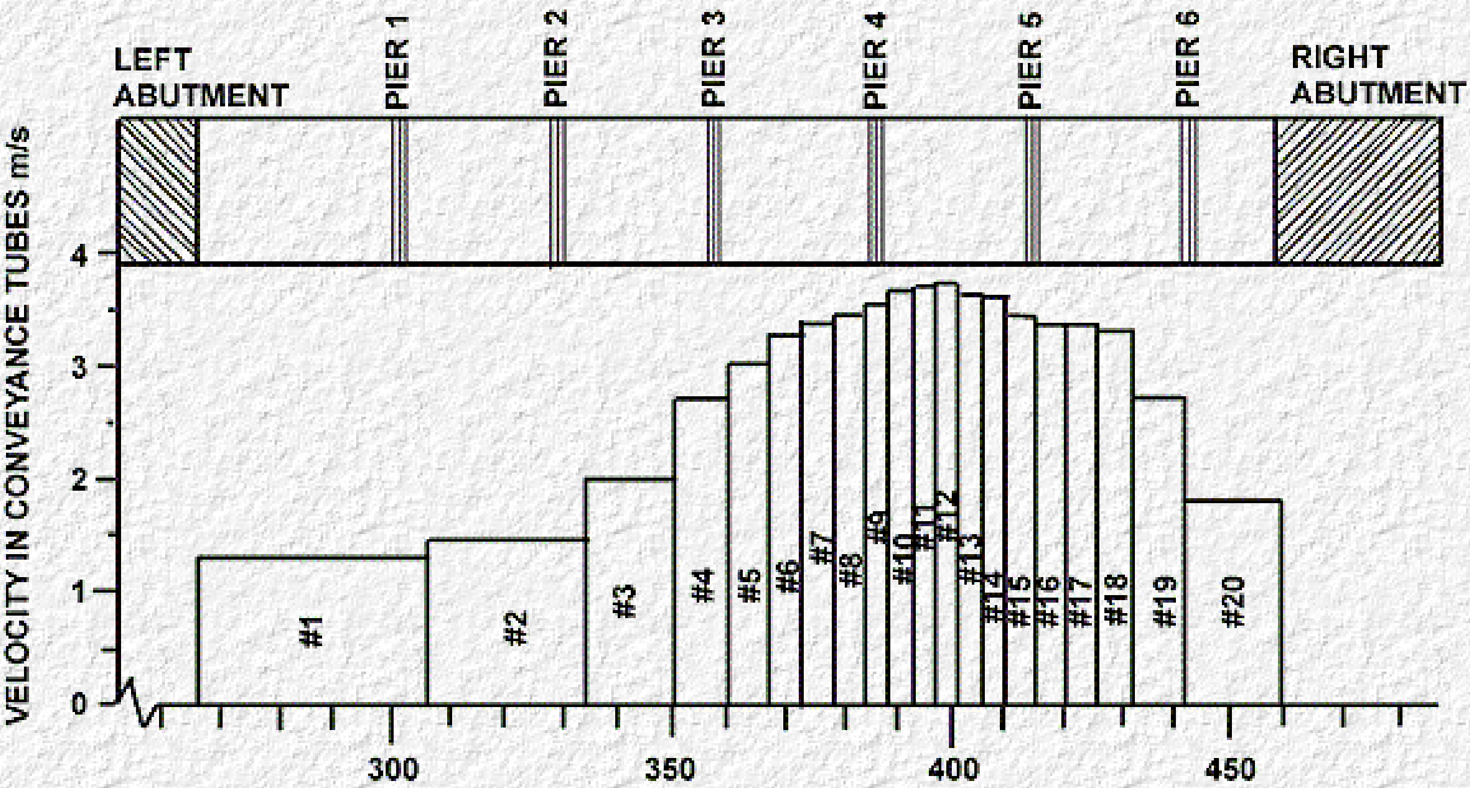

Figure 21. Velocity Distribution at Bridge Crossing

4.4.3 Step 2: Analyze Long-Term Bed Elevation Changes

Evaluation of stage discharge relationships and cross sectional data obtained from other agencies do not indicate progressive aggradation or degradation. Also, long-term aggradation or degradation are not evident at neighboring bridges. Based on these observations, the channel is relatively stable vertically, at present.

Furthermore, there are no plans to change the local land use in the watershed. The forested areas of the watershed are government-owned and regulated to prevent wide spread fire damage, and instream gravel mining is prohibited. These observations indicate that future aggradation or degradation of the channel, due to changes in sediment delivery from the watershed, are minimal.

Based on these observations, and due to the lack of other possible impacts to the river reach, it is determined that the channel will be relatively stable vertically at the bridge crossing and long-term aggradation or degradation potential is considered to be minimal. However, there is evidence that the channel is unstable laterally. This will need to be considered when assessing the total scour at the bridge.

4.4.4 Step 3: Evaluate The Scour Analysis Method

It is assumed that the components of scour will develop independently. Therefore, the contraction and local scour will be computed using the hydraulic characteristics determined from the WSPRO model. The fixed bed geometry will not be modified.

4.4.5 Step 4A: Compute the Magnitude of Contraction Scour (Main Channel)

As a precursor to the computation of contraction scour in the main channel under the bridge, it is first necessary to determine whether the flow condition in the main channel is either live-bed or clear-water. This is determined by comparing the critical velocity for sediment movement at the approach section to the average channel velocity of the flow at the approach section as computed using the WSPRO output. This comparison is conducted using the average velocity in the main channel of the approach section to the bridge. If the average computed channel velocity is greater than the critical velocity, the live-bed equation should be used. Conversely, if the average channel velocity is less than the critical velocity, the clear-water equation is applicable. The following computations are based on the quantities tabulated in Table 7.

The discharge in the main channel of the approach section is determined from the ratio of the conveyance in the main channel to the total conveyance of the approach section. By multiplying this ratio by the total discharge, the discharge in the main channel at the approach section (Q_1) is computed.

$$Q_1 = Q\,(K_1 / K_{total}) = 849.51\,m^3/s\left[\frac{19\,000}{39\,150}\right] \tag{30}$$

Q_1 = 412.28 m^3/s

The average velocity in the main channel of the approach section is determined by dividing the discharge computed in Equation 30 by the cross-sectional area of the main channel.

$$V_1 = \left(Q_1 / A_c\right) = \left[\frac{412.28}{320}\right]1.29 \;\; m/s \tag{31}$$

The average flow depth in the approach section is determined by dividing the flow area by the top width of the channel.

$$y_1 = (A_1 / TOPW) = \left[\frac{320}{121.9}\right] = 2.63 \;\; m \tag{32}$$

The channel velocity computed in Equation 31 is compared to the critical velocity of the D_{50} size for sediment movement (V_c) to determine whether the flow condition is either clear-water or live-bed.

$$V_c = 6.19\ y_1^{1/6}\ D_{50}^{1/3} \tag{33}$$

$$V_c = 6.19\ (2.63\ m)^{1/6}\ (0.002\ m)^{1/3} \tag{33a}$$

$$V_c = 0.92\ m/s$$

Since the average velocity in the main channel is greater than the critical velocity ($V_1 > V_c$), the flow condition will be live-bed. The following computations illustrate the computation of the contraction scour using the live-bed equation.

The following computation determines the mode of bed material transport and the factor k_1. All hydraulic parameters which are needed for this computation are listed in Table 7.

The hydraulic radius of the approach channel is:

$$R = \frac{A_c}{WETP} = \frac{320\ m^2}{122\ m} = 2.62\ m \tag{34}$$

Notice that the hydraulic radius of the approach is nearly equal to the average flow depth computed earlier (Equation 32). This condition indicates that the channel is wide with its width greater than 10 times the flow depth. If the width was less than 10 times the average flow depth, the channel could not be assumed to be wide and the hydraulic radius would deviate from the average flow depth.

The average shear stress on the channel bed is:

$$\tau_o = \gamma R S \tag{35}$$

$$\tau_o = (9810 N/m^3)\ (2.62\ m)\ (0.002\ m/m) = 51.4\ N/m^2 = 51.4\ Pa \tag{35a}$$

The shear velocity in the approach channel is:

$$V_* = (\tau_o/\rho)^{0.5} = (51.4/1000)^{0.5} = 0.227\ m/s \tag{36}$$

Bed material is sand with $D_{50} = 0.002$ m (2mm).

Fall velocity (ω) = 0.21 m/s from Figure 3.

Therefore:

$$\frac{V_*}{\omega} = \frac{0.227}{0.21} = 1.08 \tag{37}$$

From the above, the coefficient k_1 is determined (from the discussion for Equation 17) to be equal to 0.64 which indicates that the mode of bed material transport is a mixture of suspended and contact bed material discharge.

The discharge in the main channel at the bridge (Q_2) is determined from the ratio of conveyances for the bridge section. This procedure for obtaining the discharge is similar to the procedure used to obtain the discharge in the main channel of the approach which was previously illustrated in Equation 30.

$$Q_2 = Q\,(K_2 / K_{total}) = 849.51\,m^3/s\left[\frac{11\,300}{12\,540}\right] \tag{38}$$

Q_2 = 767.54 m^3/s

The channel widths at the approach and bridge section are given in Table 7. Therefore all parameters to determine live-bed contraction scour have been determined and Equation 17 can be employed.

$$\frac{y_1}{y_2} = \left(\frac{Q_2}{Q_1}\right)^{6/7}\left(\frac{W_1}{W_2}\right)^{k_1} \tag{39}$$

$$\frac{y_2}{y_1} = \left(\frac{767.54}{412.28}\right)^{6/7}\left(\frac{121.9}{115.9}\right)^{0.64} = 1.76 \tag{40}$$

By multiplying the above result by y_1, y_2 is determined to be equal to 4.63 m. The scour is calculated by subtracting the flow depth in the bridge (y_0) from y_2. The bridge channel flow depth (y_0) is the area divided by the top width, y_0 = 236 m^2/122 m = 1.93 m. Therefore the depth of contraction scour in the main channel is:

$$y_s = y_2 - y_0 = 4.63\text{ m} - 1.93\text{ m} = 2.7\text{ m} \tag{41}$$

This amount of contraction scour is large and could be minimized by increasing the bridge opening, providing for relief bridges in the overbank, or in some cases, providing for highway approach overtopping.

If this were the design of a new bridge, the excessive backwater (0.61 m) would require a change in the design to meet FEMA backwater requirements. The increase in backwater is obtained by subtracting the elevation given in line 264 from the elevation given in line 281 in Appendix C. However, in the evaluation of an existing bridge for safety from scour, this amount of contraction scour could occur and the scour analysis should proceed.

4.4.6 Step 4B: Compute the Contraction Scour for Left Overbank

Clear-water contraction scour will occur in the overbank area between the left abutment and the left bank of bridge opening. Although the bed material in the overbank area is soil, it is protected by vegetation. Therefore, there would be no bed-material transport into the set-back bridge opening (clear-water conditions). The subsequent computations are based on the discharge and depth of flow passing under the bridge in the left overbank. These hydraulic variables were determined from the WSPRO output and are tabulated in Table 8.

Computation of clear-water contraction scour (Equation 20).

$$y_2 = \left[\frac{0.025\ Q^2}{(D_m^{2/3} W_{contracted}^{\ 2})}\right]^{3/7} \tag{42}$$

Computation of contraction scour flow depth in left overbank area under the bridge, y_2:

$$y_2 = \left[\frac{0.025\,(81.97\ m^3/s)^2}{(0.0025\ m)^{2/3}(65.8\ m)^2}\right]^{3/7} = 1.38\ m \tag{43}$$

Computation of average flow depth in left overbank bridge section, y_0:

$$y_0 = \frac{A}{TOPW} = \frac{(57.0\ m^2)}{(68.8\ m)} = 0.83\ m \tag{44}$$

Therefore, the clear-water contraction scour in the left overbank of the bridge opening is:

$$y_s = y_2 - y_o = 1.38\ m - 0.83\ m = 0.55\ m \tag{45}$$

4.4.7 Step 5: Compute the Magnitude of Local Scour at Piers

It is anticipated that any pier under the bridge could potentially be subject to the maximum flow depths and velocities derived from the WSPRO hydraulic model (Table 9). Therefore, only one computation for pier scour is conducted and assumed to apply to each of the six piers for the bridge. This assumption is appropriate based on the fact that the thalweg is prone to shifting and because there is a possibility of lateral channel migration.

Computation of Pier Scour. The Froude Number for the pier scour computation is based on the hydraulic characteristics of conveyance tube number 12. Therefore:

$$Fr_1 = \frac{V}{(g\,y_1)^{0.5}} = \frac{3.73\ m/s}{\left[(9.81\ m/s^2)(2.84\ m)\right]^{0.5}} \tag{46}$$

$Fr_1 = 0.71$

For a round-nose pier, aligned with the flow and sand-bed material:

$$K_1 = K_2 = K_4 = 1.0 \tag{47}$$

For plane-bed condition:

$$K_3 = 1.1 \tag{48}$$

Using Equation 21:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3\ K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \tag{49}$$

$$\frac{y_s}{2.84} = 2(1)\ (1)\ (1.1)\ (1) \left(\frac{1.52m}{2.84m}\right)^{0.65} (0.71)^{0.43} \tag{50}$$

$$\frac{y_s}{2.84} = 1.26$$

y_s = 3.6 m

From the above computation the maximum local pier scour depth will be 3.6 m.

<u>Correction for Skew</u>. The above computation assumes that the piers are aligned with the flow (skew angles are less than 5°). However, if the piers were skewed to the flow greater than 5º, the value of y_s/y_1, as computed above, would need to be adjusted using K_2. The following computations illustrates the adjustment for piers skewed 10°.

$$\frac{L}{a} = \frac{12.2\ \ m}{1.52\ \ m} = 8 \tag{51}$$

K_2 can then be interpolated using an L/a of 8 and a 10° angle of attack from the correction values tabulated in Table 3. For this example, K_2=1.67. Applying this correction:

$$\frac{y_s}{2.84} = 1.67(1.26) = 2.1 \tag{52}$$

y = 6.0 m

Therefore, the maximum local pier scour depth for a pier angled 10° to the flow is 6.0 m.

<u>Discussion of Pier Scour Computations</u>. Although the estimated local pier scour would probably not occur at each pier, the possibility of thalweg shifting, which was identified in the Level 1 analysis, precludes setting the piers at different depths even if there were a substantial savings in cost. This is because any of the piers could be subjected to the worst-case scour conditions.

It is also important to assess the possibility of lateral migration of the channel. This possibility can lead to directing the flow at an angle to the piers, thus increasing local scour. Countermeasures to minimize this problem could include riprap for the channel banks both up- and downstream of the bridge, and installation of guide banks to align flow through the bridge opening.

The possibility of lateral migration precludes setting the foundations for the overbank piers at a higher elevation. Therefore, in this example the foundations for the overbank piers should be set at the same elevation as the main channel piers.

4.4.8 Step 6A: Compute the Magnitude of Local Scour at Left Abutment

Computation of Abutment Scour Using Froehlich's Equation.[58] For spill-through abutments, K_1 = 0.55. For this example, the abutments are set perpendicular to the flow; therefore, K_2=1.0. Abutment scour can be estimated using Froehlich's equation with data derived from the WSPRO output (Table 10).

The y_a value at the abutment is assumed to be the average flow depth in the overbank area. It is computed as the cross-sectional area of the left overbank cut off by the left abutment divided by the distance the left abutment protrudes into the overbank flow.

$$y_a = \frac{A_e}{L'} = \frac{264.65\ m^2}{232.80\ m} = 1.14\ m \qquad (53)$$

The average velocity of the flow in the left overbank (Figure 20) which is cut off by the left abutment is computed as the discharge cutoff by the abutment divided by the area of the left overbank cut off by the left abutment.

$$V_e = \frac{Q_e}{A_e} = \frac{148.68\, m^3/s}{264.65\, m^2} = 0.56\ m/s \qquad (54)$$

Using these parameters, the Froude Number of the overbank flow is:

$$Fr = \frac{V_e}{(g\ y_a)^{1/2}} = \frac{0.56\ m/s}{\left[\left(9.81\ m/s^2\right)\left(1.14\ m\right)\right]^{0.5}} \qquad (55)$$

Fr =0.17

Using Froehlich's equation (Equation 28):

$$\frac{y_s}{y_a} = 2.27\ K_1\ K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \qquad (56)$$

$$\frac{y_s}{1.14} = 2.27(0.55)(1.0)\left(\frac{232.8}{1.14}\right)^{0.43} (0.17)^{0.61} + 1 \qquad (57)$$

$$\frac{y_s}{1.14} = 5.17$$

$y_s = 5.9$ m

Using Froehlich's equation, the abutment scour at the left abutment is computed to be 5.9 m.

Computation of Abutment Scour Using the HIRE Equation.(13) The HIRE equation for abutment is applicable for this situation because L/y_1, as represented by L'/y_a from the previous computation, is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the abutment end which is listed in Table 11. Therefore, the Froude Number of this flow is:

$$Fr_1 = \frac{1.29 \ \ m/s}{\left[\left(9.81 \ \ m/s^2\right)\left(0.83 \ \ m\right)\right]^{0.5}} = 0.45 \tag{58}$$

Using the HIRE equation (Equation 29):

$$\frac{y_s}{0.83 \ \ m} = 4 \ \ Fr_1^{0.33} = 4(0.45)^{0.33} = 3.07 \tag{59}$$

$y_s = 2.6$ m

From the above computation, the depth of scour at the left abutment as computed using the HIRE equation, is 2.6 m.

4.4.9 Step 6B: Compute Magnitude of Local Scour at Right Abutment

The HIRE equation for abutment is also applicable for the right abutment since L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the end of the right abutment and listed in Table 12. The Froude Number of this flow is:

$$Fr_1 = \frac{2.19 \ \ m/s}{\left[\left(9.81 \ \ m/s^2\right)\left(1.22 \ \ m\right)\right]^{0.5}} = 0.63 \tag{60}$$

Using the HIRE equation:

$$\frac{y_s}{1.22\ m} = 4\ Fr_1^{0.33} = 4(0.63)^{0.33} = 3.43 \qquad (61)$$

y_s = 4.2 m

From the above computation, the depth of scour at the right abutment, as computed using the HIRE equation is 4.2 m.

Discussion of Abutment Scour Computations. Abutment scour as computed using the Froehlich equation will generally result in deeper scour predictions than will be experienced in the field. These scour depths could occur if the abutments protruded into the main channel flow, or when a uniform velocity field is cut off by the abutment in a manner that most of the returning overbank flow is forced to return to the main channel at the abutment end. For most cases, however, when the overbank area, channel banks and area adjacent to the abutment are well vegetated, scour depths as predicted with the Froehlich equation will probably not occur.

All of the abutment scour computations (left and right abutments) assumed that the abutments were set perpendicular to the flow. If the abutments were angled to the flow, a correction utilizing K_2 would be applied to Froehlich's equation or, using Figure 16 would be applied to the equation from HIRE.(13) However the adjustment for skewed abutments is minor when compared to the magnitude of the computed scour depths. For example, if the abutments for this example problem were angled 30° upstream (θ = 120°), the correction for skew would increase the computed depth of abutment scour by no more than 3 to 4 percent for the Froehlich and HIRE equation, respectively.

4.4.10 Step 7: Plot Total Scour Depth and Evaluate Design

As a final step, the results of the scour computations are plotted on the bridge cross section and carefully evaluated (Figure 22). For this example, only the computations for pier scour with piers aligned with the flow were plotted. Additionally, only the abutment scour computations reflecting the results from the HIRE equation were plotted. The topwidth of the local scour holes is suggested as 2.0 y_s.

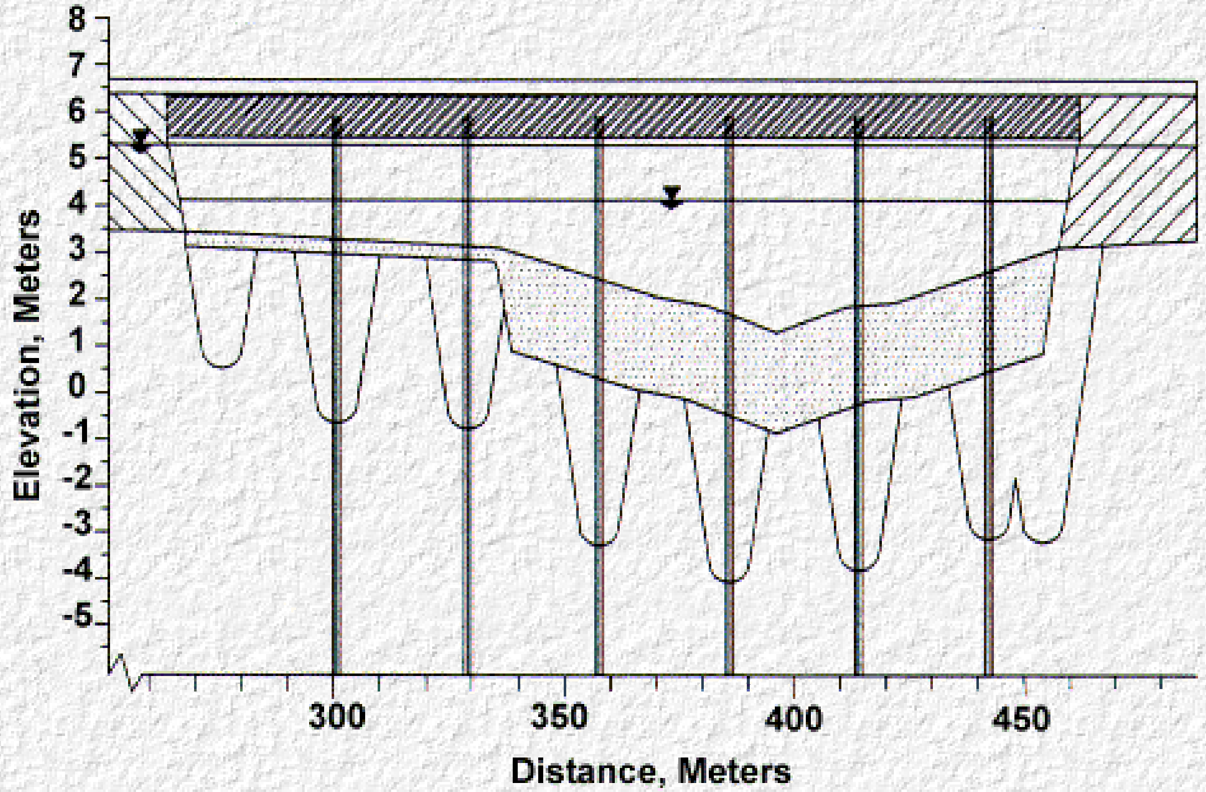

Figure 22. Plot of Total Scour for Example Problem

It is important to evaluate carefully the results of the scour computations. For example, although the total scour plot indicates that the total scour at the overbank piers is less than for the channel piers, this does not indicate that the foundations for the overbank piers can be set at a higher elevation. Due to the possibility of channel and thalweg shifting, all of the piers should be set to account for the maximum total scour. Also, the computed contraction scour is distributed uniformly across the channel in Figure 22. However, in reality this may not be what would happen. With the flow from the overbank area returning to the channel, the contraction scour could be deeper at both abutments. The use of guide banks would distribute the contraction scour more uniformly across the channel. This would make a strong case for guide banks in addition to the protection they would provide to the abutments. The stream tube velocities could be used to distribute the scour depths across this section.

The plot of the total scour also indicates that there is a possibility of overlapping scour holes between the sixth pier and right abutment, and it is not clear from where the right abutment scour should be measured, since the abutment is located at the channel bank. Both of these uncertainties should be avoided for replacement and new bridges whenever possible. Consequently, it would be advisable to set the right abutment back from the main channel. This would also tend to reduce the magnitude of contraction scour in the main channel.

The possibility of lateral migration of the channel will have an adverse effect on the magnitude of the pier scour. This is because lateral migration will most likely skew the flow to the piers. This problem has been minimized by using circular piers. An alternative approach would be to install guide banks to align the flow through the bridge opening.

A final concern relates to the location and depth of contraction scour in the main channel near the second pier and toe of the right abutment. At these locations, contraction scour in the main channel could increase the bank height to a point where bank failure and sloughing would occur. It is recommended that the existing bank lines be protected with revetment (i.e., riprap, gabions, etc.). Since the river has a history of channel migration, the bridge inspection and maintenance crews should be briefed on the nature of this problem so that any lateral migration can be identified.

4.4.11 Complete General Design Procedure

This design problem completes steps 1 through 6 of Chapter 3. The design must now proceed to steps 7 and 8 of Chapter 3, which include bridge foundation analysis and consideration of the check for superflood. This is not done for this example problem.

Go to Chapter 4, Part III

Chapter 4 : HEC 18

Estimating Scour at Bridges
Part III

[Go to Chapter 5](#)

4.5 Scour Analysis for Tidal Areas

4.5.1 Introduction

In the coastal region, scour at bridges over tidal waterways that are subjected to the effects of astronomical tides and storm surges is a combination of long-term degradation, contraction scour, local scour, and waterway instability. These are the same scour mechanisms that affect nontidal (riverine) streams. Although many of the flow conditions are different in tidal waterways, the equations used to determine riverine scour are applicable if the hydraulic conditions (depth, discharge, velocity, etc.) are carefully evaluated.[10,11]

This section presents methods and equations for determining stream stability and scour at tidal inlets, tidal estuaries, bridge crossings to islands and streams affected by tides (tidal waterways). Analysis of tidal waterways is very complex. The hydraulic analysis must consider the magnitude of the 100- and 500-year storm surge (storm tide), the characteristics (geometry) of the tidal inlet, estuary, bay or tidal stream and the effect of any constriction of the flow due to the bridge. In addition, the analysis must consider the long-term effects of the normal tidal cycles on long-term aggradation or degradation, contraction scour, local scour, and stream instability.

A storm tide or storm surge in coastal waters results from astronomical tides, wind action, and rapid barometric pressure changes. In addition, the change in elevation resulting from the storm surge may be increased by resonance in harbors and inlets, whereby, the tidal range in an estuary, bay, or inlet is larger than on the adjacent coast.

The normal tidal cycle with reversal in flow direction can increase long-term degradation, contraction scour, and local scour. If sediment is being moved on the flood and ebb tide, there may be no net loss of sediment in a bridge reach because sediments are being moved back and forth. Consequently, no net long-term degradation may occur. However, local scour at piers and abutments can occur at both the inland and ocean side of the piers and abutments and will alternate with the reversal in flow direction. If, however, there is a loss of sediment in one or both flow directions, there will then be long-term degradation in addition to local scour. Also, the tidal cycles may increase bank erosion, migration of the channel, and thus, increase stream instability.

The complexity of the hydraulic analysis increases if the tidal inlet or the bridge constrict the flow and affect the amplitude of the storm surge in the bay or estuary so that there is a large change in elevation between the ocean and the estuary or bay. A constriction in the tidal inlet can increase the velocities in the constricted waterway opening, decrease interior wave heights and tidal range, and increase the phase difference (time lag) between exterior and interior water levels. Analysis of a constricted inlet or waterway may require the use of an orifice equation rather than tidal relationships.

For the analysis of bridge crossings of tidal waterways, a three-level analysis approach similar to the approach outlined in HEC-20 is suggested.[12] Level 1 includes a qualitative evaluation of the stability of the inlet or estuary, estimating the magnitude of the tides, storm surges, and flow in the tidal waterway, and attempting to determine whether the hydraulic analysis depends on tidal or river conditions, or both. **Level 2** represents the engineering analysis necessary to obtain the velocity, depths, and discharge for tidal waterways to be used in determining long-term aggradation, degradation, contraction scour, and local scour. The hydraulic variables obtained from the Level 2 analysis are used in the riverine equations presented in previous sections to obtain total scour. Using these riverine scour equations, which are for steady-state equilibrium conditions for unsteady, dynamic tidal flow will usually result in estimating deeper scour depths than will actually occur (conservative estimate), but this represents the state of knowledge at this time for this level of analysis.

For complex tidal situations, **Level 3** analysis using physical and 2-dimensional computer models may be required. This section will be limited to a discussion of Levels 1 and 2 analyses. In Level 2 analyses, unsteady 1-dimensional or quasi 2-dimensional computer models may be used to obtain the hydraulic variables needed for the scour equations. **The Level 1, 2, and 3 approaches are described in more detail in later sections.**

The steady-state equilibrium scour equations given in previous sections of this manual are suitable for use to determine scour depths in tidal flows. As mentioned earlier, tidal flows resulting from storm surges are unsteady but no more so than unsteady riverine flows. For both cases, scour depths are conservative.

4.5.2 Overview of Tidal Processes

Glossary

Bay: A body of water connected to the ocean with an inlet.
Ebb or ebb tide: Flow of water from the bay or estuary to the ocean.
Estuary: Tidal reach at the mouth of a river.
Flood or flood tide: Flow of water from the ocean to the bay or estuary.
Littoral transport or drift: Transport of beach material along a shoreline by wave action. Also, longshore sediment transport.
Run-up, wave: Height to which water rises above still-water level when waves meet a beach, wall, etc.
Storm surge: Oceanic tide-like phenomenon resulting from wind and barometric pressure changes. Hurricane surge, storm tide.
Tidal amplitude: Generally, half of tidal range.
Tidal cycle: One complete rise and fall of the tide.
Tidal inlet: A channel connecting a bay or estuary to the ocean.
Tidal passage: A tidal channel connected with the ocean at both ends.
Tidal period: Duration of one complete tidal cycle.
Tidal prism: Volume of water contained in a tidal bay, inlet or estuary between low and high tide levels.
Tidal range: Vertical distance between specified low and high tide levels.
Tidal waterways: A generic term which includes tidal inlets, estuaries, bridge crossings to islands or between islands, inlets to bays, crossings between bays, tidally affected streams, etc.
Tides, astronomical: Rhythmic diurnal or semi-diurnal variations in sea level that result from gravitational attraction of the moon and sun and other astronomical bodies acting on the

rotating Earth.
Tsunami: Long-period ocean wave resulting from earthquake, other seismic disturbances or submarine land slides.
Waterway opening: Width or area of bridge opening at a specific elevation, measured normal to principal direction of flow.
Wave period: Time interval between arrivals of successive wave crests at a point.

Definition of Tidal and Coastal Processes. Typical bridge crossings of tidal waterways are sketched in Figure 23. From this figure, tidal flows can be defined as being between the ocean and a bay (or lagoon), from the ocean into an estuary, or through passages between islands.

Flow into (flood tide) and out of (ebb tide) a bay or estuary is driven by tides and by the discharge into the bay or estuary from upland areas. Assuming that the flow from upland areas is negligible, the ebb and flood in the bay or estuary will be driven solely by tidal fluctuations and storm surges as illustrated in Figure 24. With no inflow of water from rivers and streams, the net flow of water into and out of the bay or estuary will be nearly zero. Increasing the discharge from rivers and streams will lead to a net outflow of water to the ocean.

Hydraulically, the above discussion presents two limiting cases for evaluation of the flow velocities in the bridge reach. With negligible flow from the upland areas, the flow through the bridge opening is based solely on the ebb and flood resulting from tidal fluctuations or storm surges. Alternatively, when the flow from the streams and rivers draining into the bay or estuary is large in relationship to the tidal flows (ebb and flood tide), the effects of tidal fluctuations are negligible. For this latter case, the evaluation of the hydraulic characteristics and scour can be accomplished using the methods described previously in this chapter for inland rivers.

Bridge scour in the coastal region results from the unsteady diurnal and semi-diurnal flows resulting from astronomical tides, large flows that can result from storm surges (hurricanes, nor'easters, and tsunamis), and the combination of riverine and tidal flows. The forces which drive tidal fluctuations are, primarily, the result of the gravitational attraction of the sun and moon on the rotating earth (astronomical tides), wind and storm setup or seiching (storm surges), and geologic disturbances (tsunamis). These different forces which drive tides produce varying tidal periods and amplitudes. In general semi-diurnal astronomical tides having tidal periods of approximately 12 hours occur in the lower latitudes while diurnal tides having tidal periods of approximately 24 hours occur in the higher latitudes. In general the tidal periods for storm surges are usually much longer than the tidal period of astronomical tides.

The continuous rise and fall of astronomical tides will usually influence long-term trends of aggradation or degradation, contraction and local scour. Conversely, when storm surges or tsunamis occur the short-term contraction and local scour can be significant. Storm surges and tsunamis are a single event phenomenon which, due to their magnitude, can present a significant threat to a bridge crossing in terms of scour. The hydraulic variables (discharge, velocity, and depths) and bridge scour in the coastal region can be determined with as much precision as riverine flows. These determinations are conservative and research is needed for both cases to improve scour determinations. Determining the magnitude of the combined flows can be accomplished by simply adding riverine flood flow to the maximum tidal flow, if the drainage basin is small, or routing the design riverine flows to the crossing and adding them to the storm surge flows.

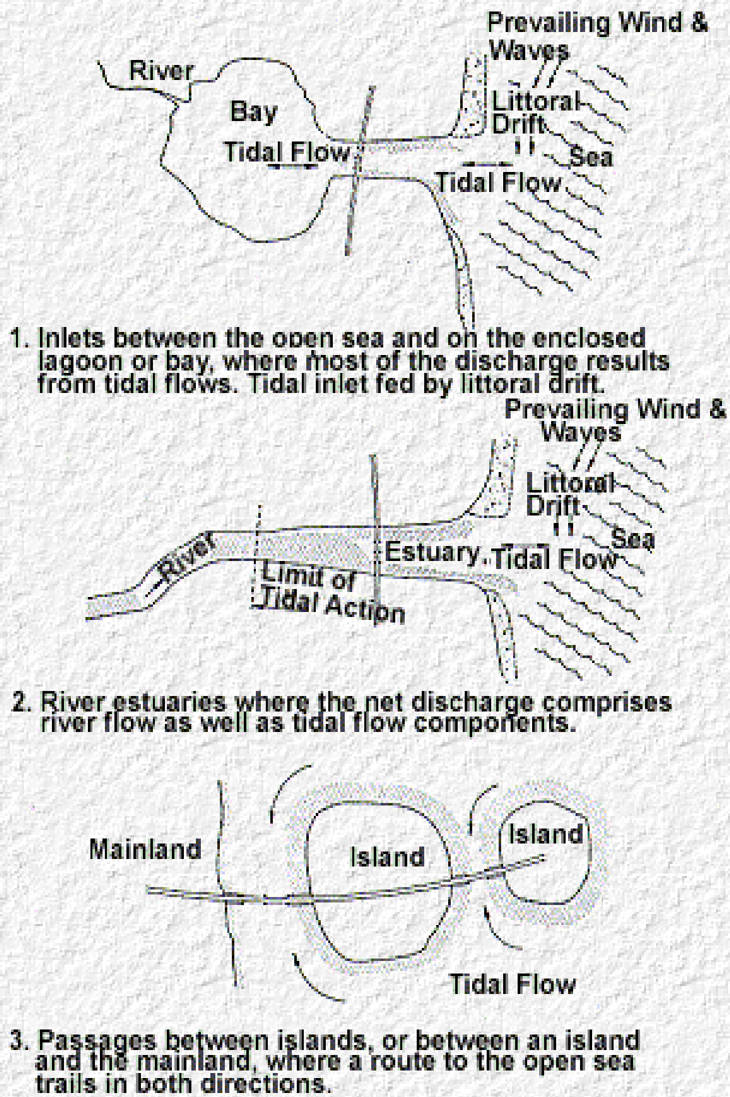

Figure 23. Types of Tidal Waterway Crossings (after Neill)[63]

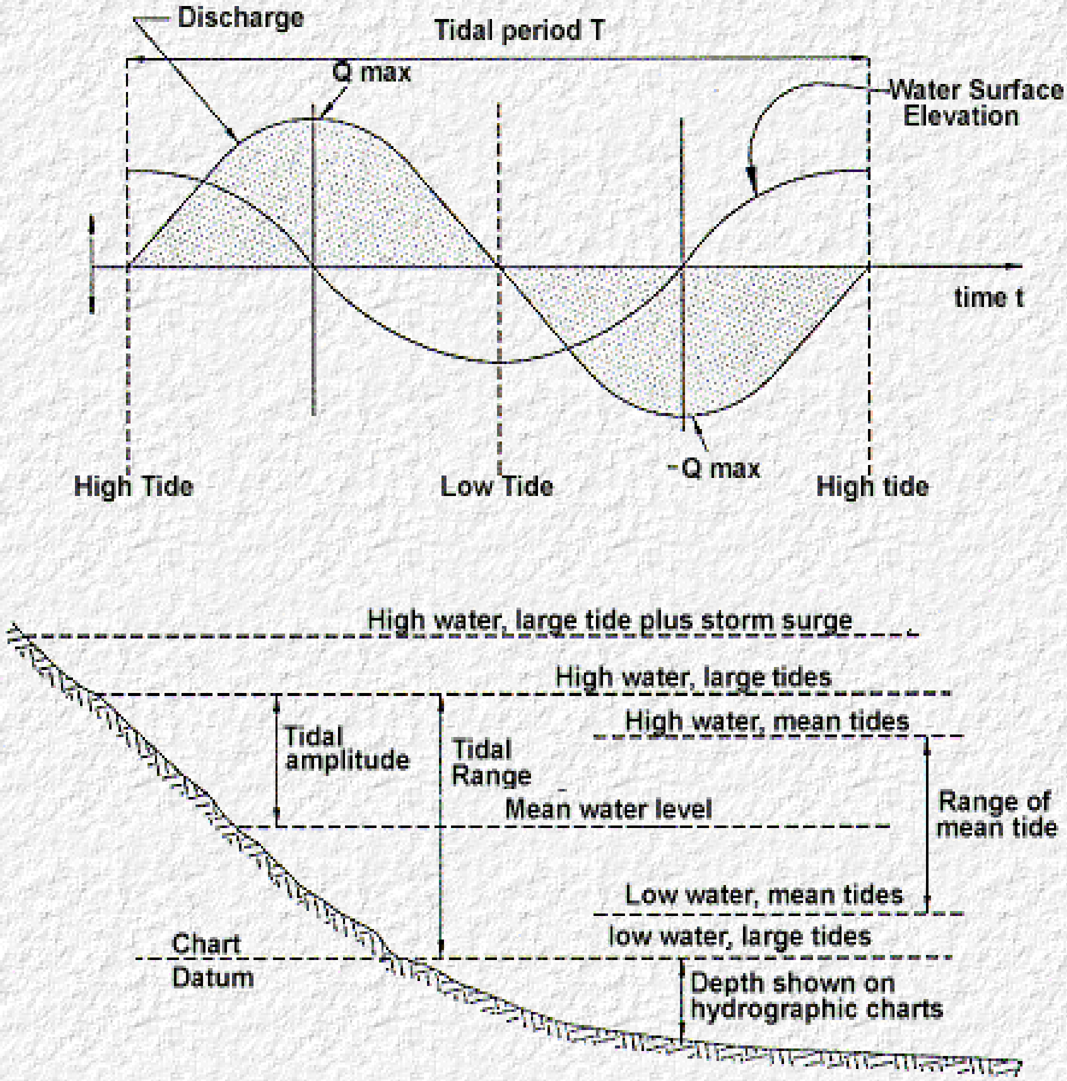

Figure 24. Principal Tidal Terms (after Neill) 63

The small size of the bed material (normally fine sand) as well as silts and clays with cohesion and littoral drift (transport of beach sand along the coast resulting from wave action) affect the magnitude of bridge scour. Mass density stratification of the water typically has a minor influence on bridge scour. Although, tidal flows are unsteady, peak flows from storm surges have durations long enough that the time is sufficient for fine sand in most coastal zones to reach scour depths determined from existing scour equations. Astronomical tides, with their daily or twice daily in- and outflows, can cause long-term degradation if there is no source of sediment except at the crossing. This has resulted in long-term degradation of 1 to 2 m per year with no indication of stopping.(64,65) Existing scour equations can predict the magnitude of this scour, but not the time history.(10,11)

Mass density stratification (saltwater wedges), which can result when the denser more saline ocean water enters an estuary or tidal inlet with significant freshwater inflow, can result in larger velocities near the bottom than the average velocity in the vertical. With careful evaluation, the correct velocity can be determined for use in the scour equations. With storm surges, mass density stratification will not normally occur. The density difference between salt and freshwater, except as it causes saltwater wedges, is not significant enough to affect scour equations. Density and viscosity differences between fresh and sediment-laden water can be much larger in riverine flows than the density and viscosity differences between salt and freshwater.

Salinity can affect the transport of silts and clays by causing them to flocculate and possibly deposit, which may affect stream stability and must be evaluated. Salinity may affect the erodibility of cohesive sediments, but this will only affect the rate of scour, not ultimate scour. Littoral drift is a source of sediment to a tidal waterway.[66,67] An aggrading or stable waterway may exist if the supply of sediment to the bridge from littoral drift is large. This will have the effect of minimizing contraction scour, and possible local scour. Conversely, long-term degradation, contraction scour and local scour can be exacerbated if the sediment from littoral drift is reduced or cut off. Evaluating the effect of littoral drift is a sediment transport problem involving historical information, future plans (dredging, jetties, etc.) for the waterway and/or the coast, sources of sediment, and other factors.

Evaluation of total scour at bridges crossing tidal waterways requires the assessment of long-term aggradation or degradation, local scour and contraction scour. Long-term aggradation or degradation estimates can be derived from a geomorphic evaluation coupled with computations of scour on live-bed contraction scour if sediment transport is changed. Such computations of long-term trends are usually driven by astronomical tide cycles. Worst-case hydraulic conditions for contraction and local scour are usually the result of infrequent tidal events such as storm surges and tsunamis.

Although the hydraulics of flow for tidal waterways is complicated by the presence of two directional flow, the basic concept of sediment continuity is valid. Consequently, a clear understanding of the principle of sediment continuity is essential for evaluating scour at bridges spanning waterways influenced by tidal fluctuations. Technically, the sediment continuity concept states that the sediment inflow minus the sediment outflow equals the time rate of change of sediment volume in a given reach. More simply stated, during a given time period the amount of sediment coming into the reach minus the amount leaving the downstream end of the reach equals the change in the amount of sediment stored in that reach.

As with riverine scour, tidal scour can be characterized by either live-bed or clear-water conditions. In the case of live-bed conditions, sediment transported into the bridge reach will tend to reduce the magnitude of scour. Whereas, if no sediment is in transport to re-supply the bridge reach (clear-water), scour depths will be larger.

In addition to sediments being transported from upland areas, sediments are transported parallel to the coast by ocean currents and wave action (littoral transport). This littoral transport of sediment serves as a source of sediment supply to the inlet, bay or estuary, or tidal passage. During the flood tide, these sediments can be transported into the bay or estuary and deposited. During the ebb tide, these sediments can be re-mobilized and transported out of the inlet or estuary and either be deposited on shoals or moved further down the coast as littoral transport (see Figure 25).

Sediment transported to the bay or estuary from the upland river system can also be deposited in the bay or estuary during the flood tide, and re-mobilized and transported through the inlet or estuary during the ebb tide. However, if the bay or estuary is large, sediments derived from the upland river system can deposit in the bay or estuary in areas where the velocities are low and may not contribute to the supply of sediment to the bridge crossing. The result is clear-water scour unless sediment transported on the flood tide (ocean shoals, littoral transport) is available on the ebb. Sediments transported from upland rivers into an estuary may be stored there on the flood and transported out during ebb tide. This would produce live-bed scour conditions unless the sediment source in the estuary was disrupted. Dredging, jetties or other coastal engineering activities can limit sediment supply to the reach and influence live-bed and clear-water conditions.

Application of sediment continuity involves understanding the hydraulics of flow and availability of sediment for transport. For example, a net loss of sediment in the inlet, bay or tidal estuary could be the result of cutting off littoral transport by means of a jetty projecting into the ocean (Figure 25). For this scenario, the flood tide would tend to erode sediment from the inlet and deposit sediment in the bay or estuary while the ensuing ebb tide would transport sediment out of the bay or estuary. Because the availability of sediment for transport into the bay is reduced, degradation of the inlet could result. As discussed later, as the cross sectional area of the inlet increases, the flow velocities during the flood tide increase, resulting in further degradation of the inlet. This can result in an unstable inlet which continues to enlarge as a result of sediment supply depletion.

From the above discussion, it is clear that the concept of sediment continuity provides a valuable tool for evaluation of aggradation or degradation trends of a tidal waterway. Although this principle is not easy to quantify without direct measurement or hydraulic and sediment continuity modeling, the principle can be applied in a qualitative sense to assess long-term trends in aggradation or degradation.

4.5.3 Level 1 Analysis

The objectives of a Level 1 qualitative analysis are to determine the magnitude of the tidal effects on the crossing, the overall long-term stability of the crossing (vertical and lateral stability) and the potential for waterway response to change.

The first step in evaluation of highway crossings is to determine whether the bridge crosses a river which is influenced by tidal fluctuations (tidally affected river crossing) or whether the highway crosses a tidal inlet, bay or estuary (tidally controlled). The flow in tidal inlets bays and estuaries is predominantly driven by tidal fluctuations (with flow reversal), whereas, the flow in tidally affected river crossings is driven by a combination of river flow and tidal fluctuations. Therefore, tidally affected crossings are not subject to flow reversal but the downstream tidal fluctuation acts as a cyclic downstream control. Tidally controlled crossings will exhibit flow reversal. The limiting case between the two types of crossings is when the magnitude of the tide is large enough to reduce the riverine discharge through the bridge opening to zero at high tide.

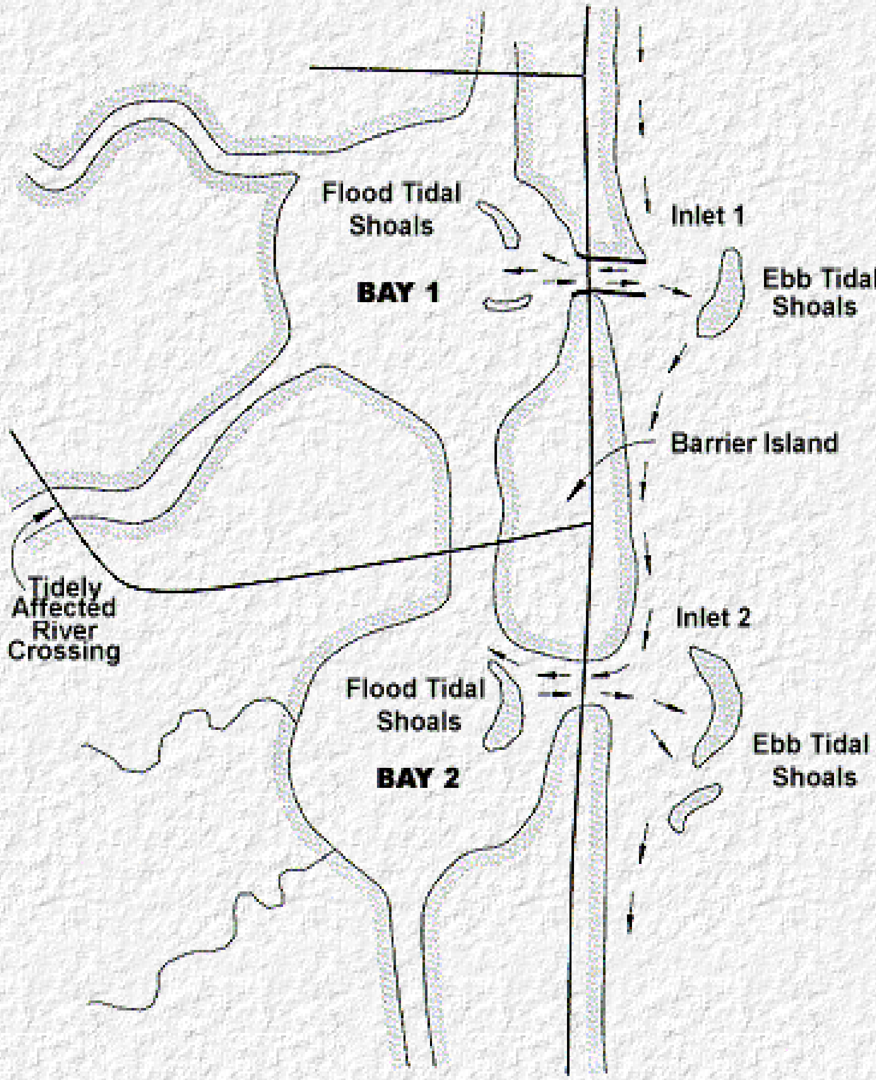

Figure 25. Sediment Transport in Tidal Inlets (after Sheppard)[66]

Tidally Affected River Crossings. Tidally affected river crossings are characterized by both river flow and tidal fluctuations. From a hydraulic standpoint, the flow in the river is influenced by tidal fluctuations which result in a cyclic variation in the downstream control of the tail water in the river estuary. The degree to which tidal fluctuations influence the discharge at the river crossing depends on such factors as the relative distance from the ocean to the crossing, riverbed slope, cross-sectional area, storage volume, and hydraulic resistance. Although other factors are involved, distance of the river crossing from the ocean can be used as a qualitative indicator of tidal influence. At one extreme, where the crossing is located far upstream, the flow in the river may only be affected to a minor degree by changes in tailwater control due to tidal fluctuations. As such, the tidal fluctuation downstream will result in only minor fluctuations in the depth, velocity, and discharge through the bridge crossing.

As the distance from the crossing to the ocean is reduced, again assuming all other factors as equal, the influence of the tidal fluctuations increases. Consequently, the degree of tail water influence on flow hydraulics at the crossing increases. A limiting case occurs when the magnitude of the tidal fluctuations is large enough to reduce the discharge through the bridge crossing to zero at high tide. River crossings located closer to the ocean than this limiting case have two directional flows at the bridge crossing, and because of the storage of the river flow at high tide, the ebb tide will have a larger discharge and velocities than the flood tide.

For the Level 1 analysis, it is important to evaluate whether the tidal fluctuations will significantly affect the hydraulics at the bridge crossing. If the influence of tidal fluctuations is considered to be negligible, then the bridge crossing can be evaluated based on the procedures outlined for inland river crossings presented previously in this document. If not, then the hydraulic flow variables must be determined using dynamic tidal flow relationships. This evaluation should include extreme events such as the influence of storm surges and design floods.

From historical records of the stream at the highway crossing, determine whether the worst-case conditions of discharge, depths and velocity at the bridge are the 100- and 500-year return period tide and storm surge, or the 100- and 500-year flood from upstream or a combination of the two. Historical records could consist of tidal and streamflow data from Federal Emergency Management Agency (FEMA), National Oceanic and Atmospheric Administration (NOAA), USACOE and USGS records; aerial photographs of the area; maintenance records for the bridge or bridges in the area; newspaper accounts of previous high tides and/or flood flows; and interviews in the local area.

If the primary hazard to the bridge crossing is from upland flood events then scour can be evaluated using the methods given previously in this circular and in HEC-20.[12] If the primary hazard to the bridge is from tide and storm surge or tide, storm surge and flood runoff, then use the analyses presented in the following sections on tidal waterways. If it is unclear whether the worst hazard to the bridge will result from a storm surge, maximum tide, or from an upland flood, it may be necessary to evaluate scour considering each of these scenarios and compare the results.

Tidal Inlets, Bays and Estuaries. **For tidal inlets, bays and estuaries, the goal of the Level 1 analysis is to determine the stability of the inlet and identify and evaluate long-term trends at the location of the highway crossing.** This can be accomplished by careful evaluation of present and historical conditions of the tidal waterway and anticipating future conditions or trends.

Existing cross-sectional and sounding data can be used to evaluate the stability of the tidal waterway at the highway crossing in terms whether the inlet, bay or estuary is increasing or decreasing in size or is relatively stable. For this analysis it is important to evaluate these data based on past and current trends. The data for this analysis could consist of aerial photographs, cross section soundings, location of bars and shoals on both the ocean and bay sides of an inlet, magnitude and direction of littoral drift, and longitudinal elevations through the waterway. It is also important to consider the possible impacts (either past or future) of the construction of jetties, breakwaters, or dredging of navigation channels.

Sources of data would be USACOE, FEMA, USGS, U.S. Coast Guard (USCG), NOAA, local Universities, oceanographic institutions and publications in local libraries. For example, a publication by Bruun, "Tidal Inlets and Littoral Drift" contains information on many tidal inlets on the east coast for the United States.[67]

A site visit is recommended to gather such data as the conditions of the beaches (ocean and bay side); location and size of any shoals or bars; direction of ocean waves; magnitude of the currents in the bridge reach at mean water level (midway between high and low tides); and size of the sediments. Sounding the channel both longitudinally and in cross section using a conventional "fish finder" sonic fathometer is usually sufficiently accurate for this purpose.

Observation of the tidal inlet to identify whether the inlet restricts the flow of either the incoming or outgoing tide is also recommended. If the inlet or bridge restricts the flow, there will be a noticeable drop in head (change in water surface elevation) in the channel during either the ebb or flood tide. If the tidal inlet or bridge restricts the flow, an orifice equation may need to be used to determine the maximum discharge, velocities and depths (see the Level 2 analysis of this section).

Velocity measurements in the tidal inlet channel along several cross sections, several positions in the cross section and several locations in the vertical can also provide useful information for verifying computed velocities. Velocity measurements should be made at maximum discharge (Q_{max}). Maximum discharge usually occurs around the midpoint in the tidal cycle between high and low tide (see Figure 24).

The velocity measurements can be made from a boat or from a bridge located near the site of a new or replacement bridge. If a bridge exists over the channel, a recording velocity meter could be installed to obtain measurements over several tidal cycles. Currently, there are instruments available that make velocity data collection easier. For example, broad-band acoustic Doppler current profiles and other emerging technologies will greatly improve the ability to obtain and use velocity data.

In order to develop adequate hydraulic data for the evaluation of scour, it is recommended that recording water level gages located at the inlet, at the proposed bridge site and in the bay or estuary upstream of the bridge be installed to record tide elevations at 15-minute intervals for at least one full tidal cycle. This measurement should be conducted during one of the spring tides where the amplitude of the tidal cycle will be largest. The gages should be referenced to the same datum and synchronized. The data from these recording gages are necessary for calibration of tidal hydraulic models such as ACES-INLET, or other unsteady 1 or 2-dimensional hydraulic flow models such as UNET, FESWMS-2D, and the TABS/FastTABS system using RMA-2V.[68,69,70,71,72] These data are also useful for calibration of WSPRO, HEC-2, or the new HEC River Analysis System (RAS) when the bridge crosses tidally affected channels.[24,42,89] A more complete description of the unsteady flow models and data requirements for model application are given in Section 4.5.4.

The data and evaluations suggested above can be used to estimate whether present conditions are likely to continue into the foreseeable future and as a basis for evaluating the hydraulics and total scour for the Level 2 analysis. A stable inlet could change to one which is degrading if the channel is dredged or jetties are constructed on the ocean side to improve the entrance, since dredging or jetties could modify the supply of sediment to the inlet. In addition, plans or projects which might interrupt existing conditions of littoral drift should be evaluated.

It should be noted that in contrast to an upland river crossing, the discharge at a tidal inlet is not fixed. In inland rivers, the design discharge is fixed by the runoff and is virtually unaffected by the waterway opening. In contrast, the discharge at a tidal inlet can increase as the area of the tidal inlet increases, thus increasing long-term aggradation or degradation and local scour. Also, as Neill points out, constriction of the natural waterway opening may modify the tidal regime and associated tidal discharge.[63]

4.5.4 Level 2 Analysis

Introduction. **Level 2 analysis involves the basic engineering assessment of scour problems at highway crossings**. At the present time, there are no suitable scour equations which have been developed specifically for tidal flows. Because of this, it is recommended that the scour equations developed for inland rivers be used to estimate and evaluate scour. However, in contrast to the evaluation of scour at inland river crossings, the evaluation of the hydraulic conditions at the bridge crossing using either WSPRO, HEC-2, or the new HEC River Analysis System (RAS) is only suitable for tidally affected crossings where tidal fluctuations result in a variable tailwater control without flow reversal.(24,42,89) Other methods, described in this section, are recommended for tidally affected and tidally controlled crossings where the tidal fluctuation has a significant influence on the tidal hydraulics.

Several methods to obtain hydraulic characteristics of tidal flows at the bridge crossing are recommended. These range from simple procedures to more complex 2-dimensional and quasi 2-dimensional unsteady flow models. The use of the simpler hydraulic procedures will be discussed and illustrated with example problems at the end of this section. An overview of the unsteady flow models which are suitable for modeling tidal hydraulics at bridge crossings will also be presented in this section.

Evaluation of Hydraulic Characteristics. The velocity of flow, depth, and discharge at the bridge waterway are the most significant variables for evaluating bridge scour in tidal waterways. Direct measurements of the value of these variables for the design storm are seldom available. Therefore, it is usually necessary to develop the hydraulic and hydrographic characteristics of the tidal waterway, estuary or bay, and calculate the discharge, velocities, and depths in the crossing using coastal engineering equations. These values can then be used in the scour equations given in previous sections to calculate long-term aggradation or degradation, contraction scour, and local scour.

Unsteady flow computer models were evaluated under a pooled fund research project administered by the South Carolina Department of Transportation (SCDOT).(73) The purpose of this study was to identify the most promising unsteady tidal hydraulic models for use in scour analyses. The study identified ACES-INLET, UNET, FESWMS-2D, and the TABS/FastTABS system using RMA-2V as being the most applicable for scour analysis.(68,69,70,71,72) The research funded by the South Carolina pooled fund project is being continued to enhance and adapt selected models so that they are better suited to the assessment of scour at bridges.

The models recommended by the pooled fund study differ in terms of their capabilities, degree of complexity, applicability and method of numerical modeling. ACES-INLET and UNET are supported by the USACOE.(68,69) ACES-INLET is restricted to analysis of tidal inlets with up to two inlets to a bay. UNET is a one-dimensional unsteady flow model and is applicable to channel networks. FESWMS-2D is an unsteady 2-dimensional finite element model developed by the USGS with support from the FHWA.(70) FESWMS-2D can be used for steady and unsteady flow analyses and incorporates structure hydraulics. RMA-2V is a 2-dimensional finite element hydrodynamic model that can be used for steady or unsteady flow analyses.(71,72)

Although these unsteady flow models are suitable for determining the hydraulic conditions, their use requires careful application and calibration. The effort required to utilize these models may be more than is warranted for many tidal situations. As such, the use of these models may be more applicable under a Level 3 analysis. However, these models could be used in the context of a Level 2 analysis, if deemed necessary, to better define the hydraulic conditions at the bridge crossing.

Alternatively, either a procedure by Neill for unconstricted waterways, or an orifice equation for constricted tidal inlets can be used to evaluate the hydraulic conditions at bridges influenced by tidal flows.(63) A step-wise procedure for using these two methods to determine hydraulic conditions and scour is presented as a prelude to the example problems presented in Section 4.5.6 and Section 4.5.7. The selection of which procedure to use depends on whether or not the inlet is constricted. In general, narrow inlets to large bays as illustrated in Figure 23 can usually be classified as constricted; whereas, estuaries, which are also depicted on Figure 23 can be classified as unconstricted. However, these guidelines cannot be construed as absolute.

The procedure developed by Neill can be used for unconstricted tidal inlets.(63) This method, which assumes that the water surface in the tidal prism is level, and the basin has vertical sides, can be used for locations where the boundaries of the tidal prism can be well defined and where heavily vegetated overbank areas or large mud flats represent only a small portion of the inundated area. Thick vegetation tends to attenuate tide levels due to friction loss, thereby violating the basic assumption of a level tidal prism. The discharges may be over estimated using this procedure if vegetation will attenuate tidal levels. In some complex cases, a simple tidal routing technique or 2-dimensional flow models may need to be used instead of this procedure.

Observation of an abrupt difference in water surface elevation during the normal ebb and flow (astronomical tide) at the inlet (during a Level 1 analysis) is a clear indication that the inlet is constricted. However, the observation of no abrupt change in water surface during astronomical tidal fluctuations does not necessarily indicate that the inlet will be unconstricted when extreme tides such as a storm surge occurs. In some cases, it may be necessary to compute the tidal hydraulics using both tidal prism and orifice procedures. Then, judgment should be used to select the worst appropriate hydraulic parameters for the computation of scour.

Velocity measurements made at the bridge site (see Level 1) can be useful in determining whether or not the inlet is constricted as well as for calibration or verification of the tidal computation procedure. Using tidal data at the time that velocity measurements were collected, computed flow depths, velocities and discharge can be compared and verified to measured values. This procedure can form a basis for determining the most appropriate hydraulic computation procedure and for adjusting the parameters in these procedures to better model the tidal flows.

Design Storm and Storm Surge. Normally, long-term aggradation or degradation at a tidal inlet or estuary are influenced primarily by the periodic tidal fluctuations associated with astronomical tides. Therefore, flow hydraulics at the bridge should be determined considering the tidal range as depicted in Figure 24 for evaluation of long-term aggradation or degradation.

Extreme events associated with floods and storm surges should be used to determine the hydraulics at the bridge to evaluate local and contraction scour. Typically, events with a return period corresponding to the 100- and 500-year storm surge and flood need to be considered. Difficulty arises in determining whether the storm surge, flood or the combination of storm surge and flood should be considered controlling.

When inland flood discharges are small in relationship to the magnitude of the storm surge and are the result of the same storm event, then the flood discharge can be added to the discharge associated with the design tidal flow, or the volume of the runoff hydrograph can be added to the volume of the tidal prism. If the inland flood and the storm surge may result from different storm events, then, a joint probability approach may be warranted to determine the magnitude of the 100- and 500-year flows.

In some cases there may be a time lag between the storm surge discharge and the stream flow discharge at the highway crossing. For this case, streamflow-routing methods such as the USACOE HEC-1 model can be used to estimate the timing of the flood hydrograph derived from runoff of the watersheds draining into the bay or estuary.(74)

For cases where the magnitude of the inland flood is much larger than the magnitude of the storm surge, evaluation of the hydraulics reduces to using the equations and procedures recommended for inland rivers. The selection of the method to use to combine flood and tidal surge flows is a matter of judgment and must consider the characteristics of the site and the storm events.

Scour Evaluation Concepts. The total scour at a bridge crossing can be evaluated using the scour equations recommended for inland rivers and the hydraulic characteristics determined using the procedures outlined in the previous sections. However, it should be emphasized that the scour equations and subsequent results need to be carefully evaluated considering other (Level 1) information from the existing site, other bridge crossings, or comparable tidal waterways or tidally affected streams in the area.

Evaluation of long-term aggradation or degradation at tidal highway crossings, as with inland river crossings, relies on a careful evaluation of the past, existing and possible future condition of the site. This evaluation is outlined under Level 1 and should consider the principles of sediment continuity. A longitudinal sonic sounder survey of a tide inlet is useful to determine if bed material sediments can be supplied to the tidal waterway from the bay, estuary or ocean. When available, historical sounding data should also be used in this evaluation. Factors which could limit the availability of sediment should also be considered.

Over the long-term in a stable tidal waterway, the quantity of sediment being supplied to the waterway by ocean currents, littoral transport and upland flows and being transported out of the tidal waterway are nearly the same. If the supply of sediment is reduced either from the ocean or from the bay or estuary, a stable waterway can be transformed into a degrading waterway. In some cases, the rate of long-term degradation has been observed to be large and deep. An estimate of the maximum depth that this long-term degradation can achieve can be made by employing the clear-water contraction scour equations to the inlet. For this computation the flow hydraulics should be developed based on the range of mean tide as described in Figure 24. It should be noted that the use of this equation would provide an estimate of the worst case long-term degradation which could be expected assuming no sediments were available to be transported to the tidal waterway from the ocean or inland bay or estuary. As the waterway degrades, the flow conditions and storage of sediments in shoals will change, ultimately developing a new equilibrium. The presence of scour resistant rock would also limit the maximum long-term degradation.

Potential contraction scour for tidal waterways also needs to be carefully evaluated using hydraulic characteristics associated with the 100- and 500-year storm surge or inland flood as described in the previous section. For highway crossings of estuaries or inlets to bays, where either the channel narrows naturally or where the channel is narrowed by the encroachment of the highway embankments, the live-bed or clear water contraction scour equations can be utilized to estimate contraction scour.

Soil boring or sediment data are needed in the waterway upstream, downstream, and at the bridge crossing in order to determine if the scour is clear-water or live-bed and to support scour calculations if clear-water contraction scour equations are used. Equation 15 or Equation 16 and the ratio of V_*/ω can be used to assess whether the scour is likely to be clear water or live bed.

A mitigating factor which could limit contraction scour concerns sediment delivery to the inlet or estuary from the ocean due to the storm surge and inland flood. A tidal surge may transport large quantities of sediment into the inlet or estuary during the flood tide. Likewise, upland floods can also transport sediment to an estuary during extreme floods. Thus, contraction scour during extreme events may be classified as live-bed because of the sediment being delivered to the inlet or estuary from the combined effects of the storm surge and flood tide. The magnitude of contraction scour must be carefully evaluated using engineering judgment which considers the geometry of the crossing, estuary or bay, the magnitude and duration of the discharge associated with the storm surge or flood, the basic assumptions for which the contraction scour equations were developed, and mitigating factors which would tend to limit contraction scour.

Evaluation of the local scour at piers can be made by using Equation 21 as recommended for inland river crossings. This equation can be applied to piers in tidal flows in the same manner as given for inland bridge crossings. However, the flow velocity and depth will need to be determined considering the design flow event and hydraulic characteristics for tidal flows.

Scour Evaluation Procedure for an Unconstricted Waterway. This method applies only when the tidal waterway or the bridge opening does not significantly constrict the flow and uses the tidal prism method as discussed by Neill.[63]

STEP 1. Determine the net waterway area at the crossing as a function of elevation. Net area is the gross waterway area between abutments minus area of the piers. It is often useful to develop a plot of the area versus elevation.

STEP 2. Determine tidal prism volume as a function of elevation. The volume of the tidal prism at successive elevations is obtained by planimetering successive sounding and contour lines and calculating volume by the average end area method. The tidal prism is the volume of water between low and high tide levels or between the high tide elevation and the bottom of the tidal waterway.

STEP 3. Determine the elevation versus time relation for the 100- and 500-year storm tides. The ebb and flood tide elevations can be approximated by either a sine or cosine curve. A sine curve starts at mean water level and a cosine curve starts at the maximum tide level. The equation for storm ebb tide that starts at the maximum elevation is:

$$y = A\ \mathrm{Cosq} + Z \qquad (62)$$

where:

y = Amplitude or elevation of the tide above mean water level, m at time t

A = Maximum amplitude of elevation of the tide or storm surge, m.
Defined as half the tidal range or half the height of the storm surge.

θ = Angle subdividing the tidal cycle, one tidal cycle is equal to 360°.

$$\theta = 360\left[\frac{t}{T}\right]$$

t = Time from beginning of total cycle, minutes

T = Total time for one complete tidal cycle, minutes

Z = Vertical offset to datum, m.

The tidal range (difference in elevation between high and low tide) is equal to 2A. One-half the tidal period is equal to the time between high and low tide. These relations are shown in Figure 24. A figure similar to Figure 24, can be developed to illustrate quantitatively the tidal fluctuations and resultant discharges.

To determine the elevation versus time relation for the 100- and 500-year storm tides, two values must be known:

- tidal range
- tidal period

As stated earlier, FEMA, USACOE, NOAA, and other federal or state agencies compile records which can be used to estimate the 100- and 500-year storm surge elevation, mean sea level elevation, and low tide elevation. These agencies also are the source of data to determine the 100- and 500-year storm tide period.

Tides, and in particular storm tides, may have different periods than the major astronomical semi-diurnal and diurnal tides which have periods of approximately 12 and 24 hours, respectively. This is because storm tides are influenced by factors other than the gravitational forces of the sun, moon and other celestial bodies. Factors such as the wind, path of the hurricane or storm creating the storm tide, fresh water inflow, shape of the bay or estuary, etc. influence both the storm tide amplitude and period.

STEP 4. Determine the discharge, velocities and depth. Neill has stated the maximum discharge in an ideal tidal estuary may be approximated by the following equation:[63]

$$Q_{max} = \frac{3.14\ VOL}{T} \qquad (63)$$

where:

Q_{max} = Maximum discharge in the tidal cycle, m^3/s
VOL = Volume of water in the tidal prism between high and low tide levels, m^3
T = Tidal period between successive high or low tides, s

A simplification of Equation 63, suggested by Chang, is to assume the tidal prism has vertical sides.[47] With this assumption, which eliminates the need to compute the volume in the tidal prism by adding the volume of successive elevations, Equation 63 becomes:

$$Q_{max} = \frac{3.14\ A_s\ H}{T} \qquad (63a)$$

where:

A_s = Surface area of the tidal prism at mean tide elevation, m^2 Distance
H = between successive high or low tides, m

In the idealized case, Q_{max} occurs in the estuary or bay at mean water elevation and at a time midway between high and low tides when the slope of the tidal energy gradient is steepest (see Figure 24).

The corresponding maximum average velocity in the waterway is:

$$V_{max} = \frac{Q_{max}}{A_c} \qquad (64)$$

where:

V_{max} = Maximum average velocity in the cross section (where the bridge will be located) at Q_{max}, m/s

A_c = Cross-sectional area of the waterway at mean tide elevation, halfway between high and low tide, m^2

It should be noted that the velocity as determined in the above equations represents the average velocity in the cross section. This velocity will need to be adjusted to estimate velocities at individual piers to account for nonuniformity of velocity in the cross section. As for inland rivers, local velocities can range from 0.9 to approximately 1.7 times the average velocity depending on whether the location in the cross section was near the banks or near the thalweg of the flow.

Neill's studies indicate that the maximum velocity in estuaries is approximately 30 percent greater than the average velocity computed using Equation 63. If a detailed analysis of the horizontal velocity distribution is needed, the design discharge could be prorated based on the conveyance in subareas across the channel cross section.

Another useful equation from Neill is:[63]

$$Q_t = Q_{max} \sin\left(360 \frac{t}{T}\right) \qquad (65)$$

where:

Q_t = Discharge at any time t in the tidal cycle, m^3/s

The velocities calculated with this procedure can be plotted and compared with any measured velocities that are available for the bridge site or adjacent tidal waterways to evaluate the reasonableness of the results.

STEP 5. Evaluate the effect of flows derived from upland riverine flow on the values of discharge, depth and velocities obtained in step 4. This evaluation may range from simply neglecting the upland flow into a bay (which is so large that the upland flow is insignificant in comparison to the tidal flows), to routing the upland flow into the bay or estuary. If an estuary is a continuation of the stream channel and the storage of water in it is small, the upland flow can simply be added to the Q_{max} obtained from the tidal analysis and the velocities then calculated from Equation 64. However, if the upland flow is large and the bay or estuary sufficiently small that the upland flow will increase the tidal prism, the upland flood hydrograph should be routed through the bay or estuary and added to the tidal prism. The USACOE HEC-1 could be used to route the flows.[74] In some instances, trial calculations will be needed to determine if and how the upland flow will be included in the discharge through the bridge opening.

STEP 6. Evaluate the discharge, velocities and depths that were determined in steps 4 and 5 above (or the following section for constricted waterways). Use engineering judgment to evaluate the reasonableness of these hydraulic characteristics. Compare these values with values for other bridges over tidal waterways in the area with similar conditions. Compare the calculated values with any measured values for the site or similar sites. Even if the measured values are for tides much lower than the design storm tides they will give an appreciation of the magnitude of discharge to be expected.

STEP 7. Evaluate the scour for the bridge using the values of the discharge, velocity and depths determined from the above analysis using the scour equations recommended for inland bridge crossings presented previously. Care should be used in the application of these scour equations, using the guidance given previously for application of the scour equations to tidal situations.

Scour Evaluation Procedure for a Constricted Waterway. The procedures given above except for steps 2 and 4 (the determination of the tidal prism, discharge, velocity and depth for unconstricted waterways) are followed. To determine these hydraulic variables when the constriction is caused by the channel and not the bridge, the following equation for tidal inlets taken from van de Kreeke or Bruun can be used.[75,76]

$$V_{max} = C_d\,(2g\,\Delta H)^{1/2} \qquad (66)$$

$$Q_{max} = A_c\,V_{max} \qquad (67)$$

where:

V_{max} = Maximum velocity in the inlet, m/s
Q_{max} = Maximum discharge in the inlet, m^3/s
C_d = Coefficient of discharge ($C_d < 1.0$)
g = Acceleration due to gravity, 9.81 m/s^2
ΔH = Maximum difference in water surface elevation between the bay and ocean side of the inlet or channel, m.
A_c = Net cross-sectional area in the inlet at the crossing, at mean water surface elevation, m^2

The difference in water surface elevation, ΔH, should be for the normal astronomical tide, the 100-year storm surge and the 500-year storm surge. The difference in height for the normal astronomical tide is used to determine potential long-term degradation at the crossing if the crossing is starved of supply sediment (i.e., construction of a jetty which cuts off littoral drift). This condition can lead to the inlet becoming unstable and enlarging indefinitely.

The coefficient of discharge (C_d) for most practical applications can be assumed to be equal to approximately 0.8. Alternatively, the coefficient of discharge can be computed using the equations given by van de Kreeke or Bruun:[75,76]

$$C_d = (1/R)^{1/2} \qquad (68)$$

where:

$$R = K_o + K_b + \frac{2g\,n^2\,L_c}{h_c^{4/3}} \qquad (69)$$

and

R = Coefficient of resistance
K_o = Velocity head loss coefficient on the ocean side or downstream side of the waterway taken as 1.0 if the velocity goes to 0
K_b = Velocity head loss coefficient on the bay or upstream side of the waterway. Taken as 1.0 if the velocity goes to 0
n = Manning's roughness coefficient
L_c = Length of the waterway, m
h_c = Average depth of flow in the waterway at mean water elevation, m

If ΔH is not known or cannot be determined easily, a hydrologic routing method developed by Chang et al., which combines the above orifice equations (Equation 66) with the continuity equation, can be used.[77] The total flow approaching the bridge crossing at any time (t) is the sum of the riverine flow (Q) and tidal flow. The tidal flow is calculated by multiplying the surface area of the upstream tidal basin (A_s) by the drop in elevation (H_s) over the specified time ($Q_{tide} = A_s\, dH_s/dt$). This total flow approaching the bridge is set equal to the flow calculated from the orifice equation.

$$Q + A_s \frac{dH_s}{dt} = C_d \; A_c \sqrt{2g \;\; \Delta H} \qquad (70)$$

where:

A_c = Bridge waterway cross-sectional area, m^2
H_s = Water surface elevation in the tidal basin upstream of the bridge, m
Q = Riverine discharge m^3/s

All other variables are as previously defined.

Equation 70 may discritized with respect to time as denoted in Equation 71 for the time interval, $\Delta t = t_2 - t_1$. Subscripts 2 and 1 represent the end and beginning of the time interval, respectively.

$$\frac{Q_1+Q_2}{2} + \frac{A_{s1}+A_{s2}}{2} \; \frac{H_{s1}-H_{s2}}{\Delta t} = C_d \left(\frac{A_{c1}+A_{c2}}{2}\right) \sqrt{2g \left(\frac{H_{s1}+H_{s2}}{2} - \frac{H_{t1}+H_{t2}}{2}\right)} \qquad (71)$$

For a given initial condition, t_1, all terms with subscript 1 are known. For $t=t_2$, the downstream tidal elevation (H_{t2}), riverine discharge (Q_2), and waterway cross-sectional area (A_{c2}) are also known or can be calculated from the tidal elevation. Only the water-surface elevation (H_{s2}) and the surface area (A_{s2}) of the upstream tidal basin remain to be determined. Because surface area of the tidal basin is a function of the water-surface elevation, the elevation of the tidal basin at time t_2 (H_{s2}) is the only unknown term in Equation 71, and this term can be determined by trial-and-error to balance the values on the right and left sides.

Chang et al. suggest the following steps for computing the flow:[77]

- Step 1. Determine the period and amplitude of the design tide(s) to establish the time rate of change of the water-surface on the downstream side of the bridge.

- Step 2. Determine the surface area of the tidal basin upstream of the bridge as function of elevation by planimetering successive contour intervals and plotting the surface area vs. elevation.

- Step 3. Plot bridge waterway area vs. elevation.

- Step 4. Determine the quantity of riverine flow that is expected to occur during passage of the storm tide through the bridge.

- Step 5. Route the flows through the contracted waterway using Equation 71, and determine the maximum velocity of flow.

An example problem using the procedure developed by Chang, et al. is presented in an example problem using a case study in Appendix D.[77] A spreadsheet and a computer program developed by Maryland SHA is given to aid in using this method.

Using the tidal hydraulics determined as described above for constricted inlets, the scour computations can proceeded according to steps 5, 6, and 7 presented previously for the unconstricted waterway.

Tidal Calculations Using Unsteady Flow Models. Alternatively, the tidal hydraulics at the bridge can be determined using one of several unsteady flow models in lieu of either Neill's procedure, the orifice equation or Chang's procedure. A brief overview these models is presented below. This information was derived from a pooled fund study (HPR552) administered by the SCDOT.[73] All quotes presented in this section are from the final report documenting the first phase of this study.

ACES is an acronym for the Automated Coastal Engineering System and was developed by the USACOE in an effort to incorporate many of the various computational procedures typically needed for coastal engineering analysis into an integrated, menu-driven user environment.[68] There are seven separate computation modules for wave prediction, wave theory, littoral processes and other useful modules. One such module denoted as ACES-INLET is a spatially integrated numerical model for inlet hydraulics. This module can be used to determine discharges, depths and velocities in tidal inlets with up to two inlets connecting a bay to the ocean. This module can be used in place of, or in addition to, the procedures given in steps 3 and 4, above, for tidal inlets. **ACES-INLET is applicable only where the project site is at or very near the inlet throat (i.e., for bridges crossing inlets) (Figure 24).**

The pooled fund study states:[73]

> "*ACES-Inlet is simple and easy to use. A minimum of data are required and the menu-driven environment makes user input straightforward. The primary limitation of the model is its reliance on numerous empirical coefficients. In addition to requiring keen judgment on the part of the user, the empirical relations greatly oversimplify the inlet dynamics. Model results can be regarded as rough approximations, useful for reconnaissance-level investigations.*"

Other modules incorporated into ACES may be useful in evaluating tidal highway crossings. These modules can be used to estimate wave and tidal parameters, littoral drift, wave run-up and other aspects of tidal flow which could influence the design or evaluation of bridge crossings over tidal inlets connecting bays to the ocean.

UNET is a 1-dimensional unsteady flow model.[69] Although simpler to use than more complex 2-dimensional models, UNET can model networks of open channels, and bifurcations and flow around islands. According to the pooled fund study:

> *"UNET is extremely flexible in modeling of channel networks, storage areas, bifurcations, and junctions. Both external boundaries (hydrographs, stage hydrographs) and internal boundary conditions (gated and uncontrolled spillways, bridges, culverts, and levee systems) can be included. UNET uses a modified HEC-2 file format to facilitate data entry and UNET can use the HEC-DSS database for input and output."*

According to the pooled fund study, the advantages and limitations of UNET are:

> *"UNET uses an efficient implicit numerical formulation solution techniques. Of the reviewed unsteady 1-dimensional flow models, UNET is the only model which intrinsically evaluated bridges, culverts, and embankment overtopping.... Although UNET does not simulate flow separation (2-D), off-channel storage (ineffective flow areas) can be used to represent these areas. The primary limitation of this model is the exclusion of wind effects."*

FESWMS-2DH is a 2-dimensional unsteady flow model developed by the USGS and FHWA.[70] This model uses a finite element numerical simulation and has options for simulation of steady or unsteady flow over highway embankments and through culverts. The critique of FESWMS-2DH in the pooled fund study states:

> *"The options for weir flow and culvert flow are particularly well suited to highway application. The variable friction formulation permits realistic modeling of floodplains. The GKS format of the model output allows for storage of graphical data and its use by other programs.... FESWMS-2DH has limitations similar to those of other 2- models, e.g. inability to simulate stratified flows or complex near-field phenomena where vertical velocities are not negligible. The relative complexity of the model (as compared to 1-D models) requires some expertise for model setup and use."*

RMA-2V is a widely used 2-dimensional unsteady flow model which uses a finite element numerical procedure.[71,72] The model is incorporated into the TABS/FastTABS user interface which provides additional applications including STUDH which, when linked with RMA-2V, modifies the geometry of the waterway using computations of sediment erosion, sedimentation and transport during each time step of the hydrodynamic model. The critique of RMA-2V in the pooled fund study states:

> *"RMA-2V and the TABS/FastTABS system offer a rigorous 2-D solution to the shallow water equations coupled with sediment transport capabilities and advanced pre/post processors. The finite element spatial discretization is accurate and can easily represent complex physical systems. Other capabilities include simulation of wetting and drying elements and flow control structures..."*

All of the unsteady flow models described above can be used on a 486 or faster personal computer. Of the four unsteady models, ACES and UNET are significantly simpler than either FESWMS or RMA-2V. Because of this, ACES and UNET can be considered to be more adaptable to Level 2 type analysis due to their relative simplicity. Although FESWMS and RMA-2V can be used as part of an advanced Level 2 analysis, their use is more consistent with a Level 3 analysis. As indicated earlier, efforts to enhance and improve these models so that they better support highway applications are ongoing. Future enhancements and versions of these models will likely provide for simpler application and better estimates of the hydraulic conditions which influence scour.

Data Requirements for Hydraulic Model Verification. Whenever a hydraulic model is employed, it is necessary to tune the model to insure that the results will adequately represent the flow conditions which are likely to occur during an extreme event. Because of this, any model, including WSPRO, HEC-2, and the new HEC River Analysis System (RAS) should be verified against actual data. For inland rivers systems which are modeled using WSPRO, HEC-2, or the new HEC River Analysis System (RAS) model verification is reasonably straightforward. Known discharges and water surface elevations are used to adjust the downstream boundary conditions and resistance parameters until a close agreement between measured data and model output is obtained. Although similar, model verification using unsteady flow models is more difficult due to the unsteady nature of the flow. The following paragraphs discuss data needs for model verification of unsteady flow models.

Ideally, synoptic measurements of the following data are required to validate hydraulic modeling using any of the above mentioned unsteady flow models.:

- Tidal elevations in the ocean and back bay locations.
- Velocity measurements are needed in the inlet throat as well as at proposed project sites.
- Boundary condition data for any back-bay, open-water boundaries; these data may be elevation, velocity, discharge, or any combination of these parameters.
- Wind speed and direction if wind energy influences in the tidal system.

The above data may be available from previous studies of the tidal system (for example, USACOE or NOAA studies) or may be collected for a specific project.

4.5.5 Level 3 Analysis

As discussed in HEC-20, Level 3 analysis involves the use of physical models or more sophisticated computer models for complex situations where Level 2 analysis techniques have proven inadequate.(12) In general, crossings that require Level 3 analysis will also require the use of qualified hydraulic engineers. Level 3 analysis by its very nature is specialized and beyond the scope of this manual.

4.5.6 Example Problem Number 1

In this example problem, the discharge, velocity, depths, and scour are to be determined for an existing bridge across a tidal estuary as part of an ongoing scour evaluation. The bridge is 818.39 m long, has vertical wall abutments and 16 3.66-m diameter circular piers supported on piles. Neither the bridge or the tidal waterway constricts the flow.

For this evaluation, the bridge maintenance engineer has expressed concern about observed scour at one of the piers. This pier is located where the velocities at the pier are approximately 30 percent greater than the average velocities. The water depth at the pier referenced to mean sea level is 3.75 m. The actual depth of flow at the pier will need to be increased to account for additional water depth caused by the storm surge for the computation of pier scour.

Level 1 Analysis

a. Level 1 analysis has determined that the 100- and 500-year return period tidal storm surge discharge, velocity and depths are much larger than those from upland runoff. There is minimal littoral drift and historical tides are low. From FEMA, the storm surge tidal range for the 100-year return period is 2.19 m and 500-year return period is 2.87 m. Measured maximum velocity in the waterway at mean water level for a high tide of 0.67 m was only 0.21 m/s.

Sonic soundings in the waterway indicate there is storage of sediment in the estuary directly inland from the bridge crossing. This was determined by observing that the elevation of the bed of the waterway at the bridge site was lower than the elevation of the bottom of the estuary further inland. Although no littoral drift is evident, there is storage of sediment at the mouth of the estuary between the ocean and the bridge crossing.

b. Stability of the estuary and crossing was evaluated by examination of the periodic bridge inspection reports which included underwater inspections by divers, evaluation of historical aerial photography, and depth soundings in the estuary using sonic fathometers. From this evaluation it was determined that the planform of the estuary has not changed significantly in the past 30 years. These observations indicate that the estuary and bridge crossing has been laterally stable.

Evaluation of sounding data at the bridge indicates that there has been approximately 1.52 m of degradation at the bridge over the past 30 years; however, the rate of degradation in the past 5 years has been negligible. Underwater inspections indicted that local scour around the piers is evident.

c. A search of FEMA, USACOE, and other public agencies for flood and storm surge data was conducted. These data will be discussed under the Level 2 analysis.

d. Grain size analysis of the bed material indicates that the bed of the estuary is composed of fine sand with a D_{50} of approximately 0.27 mm (0.000 27 m).

e. Velocities measured at Q_{max} during a large tide indicated that the maximum velocity in the bridge section was approximately 30 percent greater than the average velocity.

Level 2 Analysis

STEP 1. A plot of net waterway area as a function of elevation is given in Figure 26. Net waterway area is the average area at the bridge crossing less the area of the piers.

STEP 2. A plot of volume of the tidal prism as a function of elevation is also presented in Figure 26. It was developed by planimetering the area of successive sounding and contour lines and multiplying the average area by the vertical distance between them.

STEP 3. A synthesized storm surge for the 100- and 500-year return period was developed and is presented in Figure 26. It was obtained as follows:

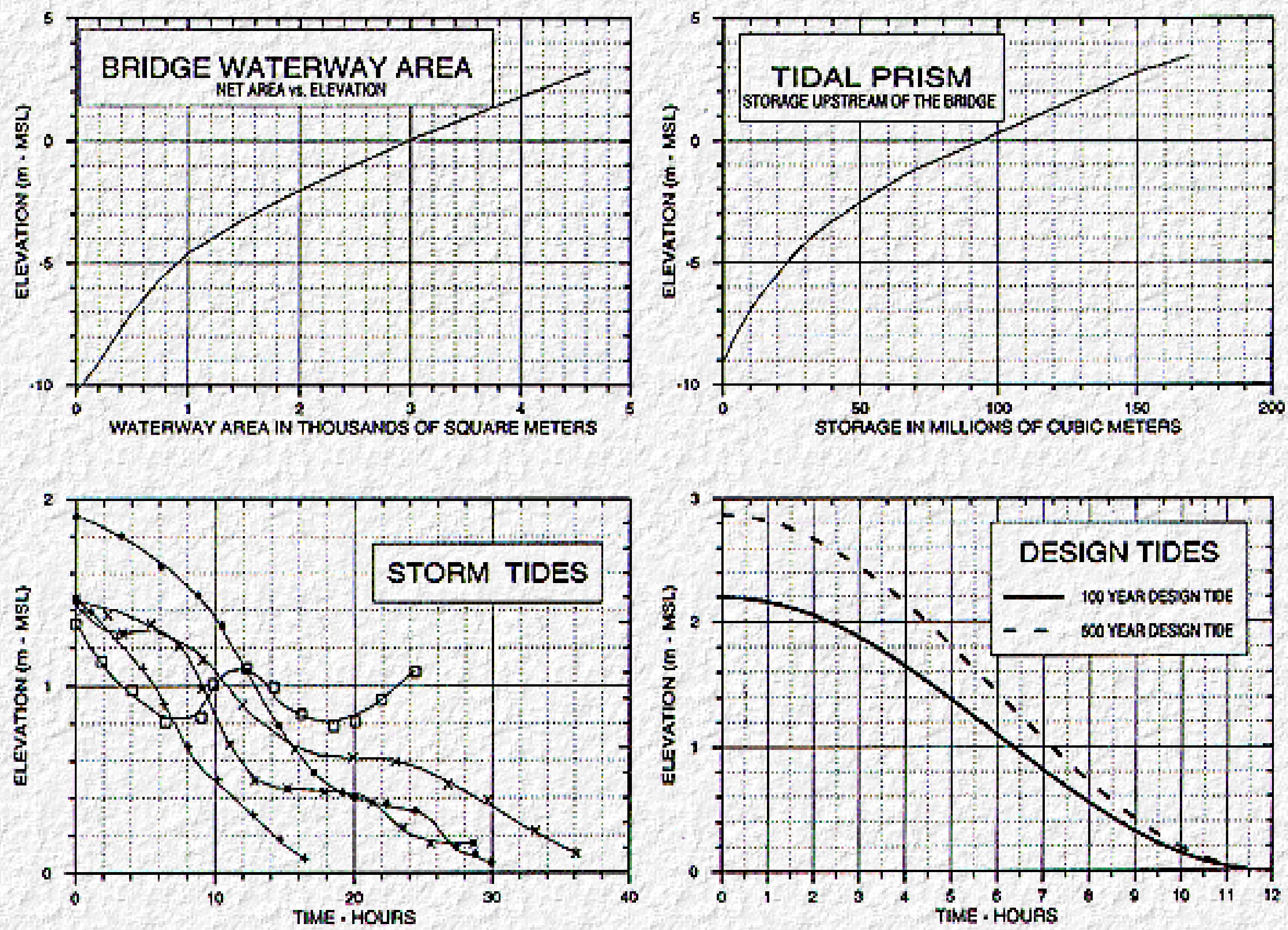

Figure 26. Tidal Parameters for Example Problem 1

An idealized tidal cycle for one half the tidal period, beginning at high tide was developed using the cosine equation (Equation 68). This plot can be used to develop an idealized tidal cycle for any waterway. Tidal range and period are needed to use the idealized tide cycle to develop a synthesized tidal cycle for this waterway.

The tidal ranges were obtained from a FEMA coastal flood insurance study during the Level 1 analysis (Table 13).

Table 13. Tidal Ranges Derived from FEMA Flood Study

Return Period (yr.)	High Tide (m)	Low Tide (m)
100	2.19	0
500	2.87	0

The tidal period is more difficult to determine because it is affected by more than the gravitational attraction of the moon and sun. At this waterway location, the direction of the storm and the characteristics of the estuary affected the tidal period. To determine the tidal period, major storm tides were plotted, as the fourth plot in Figure 26. From a study of these major storm tides a period of 12 hours was selected as being a conservative estimate of the time from flood (high) to ebb (low) tide. Tidal period (T) is then 24 hours.

STEP 4. Using the data developed in steps 1 to 3 and the equations given previously the maximum tidal discharge (Q_{max}) and maximum average tidal velocity (V_{max}) are calculated. The values used in the calculations are given in Table 14.

STEP 5. The 100- and 500-year return period peak upland flow into the estuary was obtained from a USGS flood frequency study. These values are also given in Table 14.

Average flow depths can be determined by dividing the flow area as listed in Table 14 by the channel width (818.39 m). Therefore the average flow depth for the 100- and 500-year event are 4.43 and 4.65 m, respectively.

The volume of the runoff from the 100- and 500-year upland flow hydrograph is very small in comparison to the storage volume in the estuary. In this case, adding the peak discharge to the maximum tidal discharge will be a conservative estimate of the maximum discharge and maximum average velocity in the waterway. If the upland inflow into the estuary had been large, the flood could be routed through the estuary using standard hydrologic modeling techniques.

STEP 6. A comparison of the calculated velocities with the measured velocities indicate that they are reasonable. Simply adding the peak inflow from the upland runoff results in a conservative estimate of the average velocity. Therefore, the discharge and velocities given in Table 14 are acceptable for determining the scour depths. However, the average velocity will have to be adjusted for the nonuniformity of flow velocity in the vicinity of the bridge to obtain the velocities for determining local scour at the piers.

STEP 7. Calculate the components of total scour using the information collected in the Level 1 and Level 2 analyses.

Table 14. Design Discharge and Velocities

	100-Year Storm Tide	500-Year Storm Tide
Maximum storm tide elevation, m	2.19	2.87
Mean storm tide elevation, m	1.10	1.44
Low storm tide elevation, m	0.0	0.0
Tidal prism volume (millions of cubic meters) Figure 26	46.40	60.80
Net waterway area at mean storm tide elevation (A_c), m^2	3620	3809
Tidal period, h	24.0	24.0
Q_{max} Tidal-m^3/s (Equation 63)	1686.3	2209.6
V_{max} Tidal-m/s (Equation 64)	0.47	0.58
Upland peak runoff m^3/s	141.03	224.29
Q_{max} (Tidal plus runoff) m^3/s	1828.73	2433.90
V_{max} (Tidal plus runoff) m/s ($V_{max} = Q_{max}/A_c$)	0.50	0.64

Average flow depth - A_c/width, m	4.42	4.65

Long-Term Aggradation/Degradation. The Level 1 analysis indicates that the channel is relatively stable at this time. However, there is an indication that over the past 30 years the channel has degraded approximately 1.52 m. Therefore, for this evaluation, an estimate of long-term degradation of approximately 1.52 m for the future will be assumed.

Contraction Scour. Contraction scour depends on whether the flow will be clear-water or live-bed. Equation 15 is used to determine the critical velocity for the 100-year hydraulics.

$$v_c = 6.19(4.42)^{1/6}(0.00027)^{1/3} = 0.5 \text{ m/s} \quad (72)$$

This indicates that the 100-year storm surge combined with the inland flow may result in velocities greater than or equal to the critical velocity; therefore, contraction scour will most likely be live-bed. This conclusion is made considering that velocities in excess of the average velocity will be expected due to the nonuniformity of the velocity in the bridge opening, as determined during the Level 1 analysis.

Applying the modified live-bed contraction scour equation, it is noted that the ratio of discharges is equal to unity. Therefore, the contraction scour will be influenced by the contraction resulting from the bridge piers reducing the flow width at the bridge crossing. Using Equation 17, and assuming that the mode of sediment transport is mostly suspended load (k_1=0.69), the estimate of live-bed contraction scour for the 100-year event is:

$$\frac{y_2}{4.42} = \left[\frac{818.39}{759.84}\right]^{0.69} = 1.05 \quad (73)$$

Therefore, the contraction scour for the 100-year event is approximately 0.22 m. Recomputation for the 500-year event with an average flow depth of 4.65 m results in an estimate of contraction scour of approximately 0.23 m.

Local Scour at Piers. The hydraulic analysis estimates average velocities in the bridge cross section only. Because of this, an estimate of the maximum velocity at the bridge pier is made to account for non-uniform velocity in the bridge cross section. The average velocity will be increased by 30 percent since velocities for normal flows (Level 1) indicated that the maximum velocity were observed to be approximately 30 percent greater than the average. Therefore the maximum velocity for the 100- and 500-year event are 0.65 and 0.83 m/s, respectively.

K_1, K_2, and K_4 equal 1.0. K_3 will be equal to 1.1 since the bed condition at the bridge is plane-bed. The depth of flow at the pier for the 100- and 500-year storm surge is determined by adding the mean storm tide elevation from Table 14 to the flow depth at the pier referenced to mean sea level. From this, y_1 will be equal to 4.85 and 5.19 m for the 100- and 500-year storm surge, respectively.

Applying the Equation 21 for the 100-year event:

$$\frac{y_s}{4.85} = 2.0(1.0)(1.0)(1.1)(1.0)\left[\frac{3.66}{4.85}\right]^{0.65}(0.094)^{0.43} = 0.66 \quad (74)$$

From the above equation, the local scour at the piers is 3.2 m. Considering the 500-year event, local pier scour is 3.6 m.

4.5.7 Example Problem Number 2

This problem presents a Level 2 analysis of a bridge over a tidal inlet where the waterway constricts the flow and illustrates how depletion of sediment supplied to the tidal inlet can result in a continual and severe long-term degradation. The length of the inlet is 457.2 m, the width of the bridge opening and inlet is 124.97 m, Manning's n is 0.03, depth at mean water level is 6.1 m and area A_c is 761.81 m^2. The D_{50} of the bed material is 0.30 mm and the D_m (1.25 D_{50}) is 0.375 mm (0.000 375 m).

From tidal records, the long-term average difference in elevation from the ocean to the bay, through the waterway, averaged for both the flood and ebb tide is 0.183 m. The difference in elevation for the 100-year storm surge is 0.549 m and for the 500-year storm surge is 0.884 m.

a. Determine the long-term potential degradation that may occur because construction of jetties has cut off the delivery of bed sediments from littoral drift to the inlet.

For this situation, long-term degradation can be approximated by assuming clear-water contraction scour and using the average difference in water surface between the ocean and bay for the hydraulic computation using the orifice equations (Equation 66, Equation 67, Equation 68, and Equation 69).

Using Equation 69, determine R:

$$R = 0.7 + 1.0 + \frac{2(9.81)(0.03)^2 \;\; 457.2}{(6.10)^{4/3}} \qquad (75)$$

R = 2.42

From Equation 68, determine C_d:

$$C_d = \left[\frac{1}{2.42}\right]^{1/2} \qquad (76)$$

C_d = 0.643

Using Equation 66, determine V_{max}:

$$V_{max} = 0.643\,[(2)\,(9.81)(0.183)]^{0.5} \qquad (77)$$
$$V_{max} = 1.22 \text{ m/s}$$

Using Equation 67, determine Q_{max}:

$$Q_{max} = V_{max}\,A_c = 1.22(761.81) \qquad (78)$$

$$Q_{max} = 929.41 \text{ m}^3/\text{s}$$

Potential long-term degradation for fine bed material is determined using the clear-water contraction scour equation (Equation 20):

$$y = \left[\frac{0.025\ (929.41)^2}{0.000\ \ 375^{2/3}(124.97)^2} \right]^{3/7} = 10.94\ \ \mathrm{m} \tag{79}$$

y_s =10.94 - 6.10 = 4.84 m

Discussion of Potential Long-Term Degradation. This amount of scour would occur in some time period that would depend on the amount of sediment that was available from the bay and ocean side of the waterway to satisfy the transport capacity of the back and forth movement of the water from the flood and ebb tide. Even if there was no sediment inflow into the waterway, the time it would take to reach this depth of scour is not known.

To determine the length of time would require the use of an unsteady tidal model, and conducting a sediment continuity analysis. Using a tidal model and sediment continuity analysis, calculate the amount of sediment eroded from the waterway during a tidal cycle and determine how much degradation this will cause. Then using this new average depth, recalculate the variables and repeat the process. Knowing the time period of the tidal cycle, then the time to reach a scour depth of 4.84 m could be estimated for the case of no sediment inflow into the waterway. Estimates of sediment inflow in a tidal cycle could be used to determine the time to reach the above estimated contraction scour depth when there is sediment inflow.

When the long-term degradation reaches 4.84 m, the scouring may not stop. The reason for this is that the discharge in the waterway is not limited, as in the case of inland rivers, but depends on the amount of flow that can enter the bay in a half tidal cycle. As the area of the waterway increases the flood tide discharge increases because, as an examination of Equation 66 and Equation 67 show the velocity does not decrease. There may be a slight decrease in velocity because the difference in elevation from the ocean and the bay might decrease as the area increases. However, R in Equation 69 decreases with an increase in depth.

Although the above discussion would indicate that long-term degradation would increase indefinitely, this is not the case. As the scour depth increases there would be changes in the relationship between the incoming tide and the tide in the bay or estuary, and also between the tide in the bay and the ocean on the ebb tide. This could change the difference in elevation between the bay and ocean. At some level of degradation the incoming or out-going tides could pick up sediment from either the bay or ocean which would then satisfy the transport capacity of the flow. Also, there could be other changes as scour progressed, such as accumulation of larger bed material on the surface (armor) or scour resistance rock which would decrease or stop the scour.

In spite of these limiting factors, the above problem illustrates the fact that with tidal flow, in contrast to river flow, as the area of the cross section increases from degradation there is no decrease in velocity and discharge.

b. Determine V_{max}, Q_{max} for the 100-year storm surge and a depth of 6.1 m.

The values of R and C_d do not change.

$$V_{max} = 0.643\ (2g\ 0.549)^{0.5} \tag{80}$$

$$V_{max} = 2.11\ m/s$$

$$Q_{max} = 1607.42\ m^3/s$$

These values or similar ones depending on the long-term scour depth, would be used to determine the local scour at piers and abutments using equations given previously. These values could also be used to calculate contraction scour resulting from the storm surge.

Go to Chapter 5

Chapter 5 : HEC 18

Evaluating the Vulnerability of Existing Bridges to Scour

Go to Chapter 6

5.1 Introduction

Existing bridges over water subject to scour should be evaluated to determine their vulnerability to floods and whether they are scour vulnerable (Technical Advisories T5140.23, 1991).[6] This assessment or evaluation should be conducted by an interdisciplinary team of hydraulic, geotechnical, and structural engineers who can make the necessary engineering judgments to determine:

1. Priorities for making bridge scour evaluations;
2. The scope of the scour evaluations to be performed in the office and in the field;
3. Whether or not a bridge is vulnerable to scour damage; i.e., whether the bridge is a low risk, scour susceptible, or scour-critical bridge;
4. Which alternative scour countermeasures would be applicable to make a bridge less vulnerable;
5. Which countermeasure(s) is most suitable and cost-effective for a given bridge;
6. Priorities for installing scour countermeasures;
7. Monitoring and inspection schedules for scour-critical bridges; and
8. Interim procedures to protect the bridge and the public until the bridge is repaired, replaced or until suitable long-term countermeasures are in place.

The factors to be considered in a scour evaluation require a broader scope of study and effort than those considered in a bridge inspection. The major purpose of the bridge inspection is to identify changed conditions which may reflect an existing or potential problem. The scour evaluation is an engineering assessment of the risk of what might possibly happen in the future and what steps can be taken immediately to eliminate or minimize the risk.

5.2 The Evaluation Process

The following approach is recommended for the development and implementation of a program to assess the vulnerability of existing bridges to scour:

STEP 1. Screen all bridges over waterways into five categories:

1. low risk,
2. scour-susceptible,

3. scour-critical,
4. unknown foundations, or
5. tidal.

Bridges which are particularly vulnerable to scour failure should be identified immediately and the associated scour problem addressed. These particularly vulnerable "scour-susceptible" bridges are:

a. Bridges currently experiencing scour or that have a history of scour problems during past floods as identified from maintenance records and experience, bridge inspection records, etc.

b. Bridges over erodible streambeds streams with design features that make them vulnerable to scour, including:

- Piers and abutments designed with spread footings or short pile foundations;
- Superstructures with simple spans or nonredundant support systems that render them vulnerable to collapse in the event of foundation movement; and
- Bridges with inadequate waterway openings or with designs that collect ice and debris. Particular attention should be given to structures where there are no relief bridges or embankments for overtopping, and where all water must pass through or over the structure.

c. Bridges on aggressive streams and waterways, including those with:

- Active degradation or aggradation of the streambed;
- Significant lateral movement or erosion of streambanks;
- Steep slopes or high velocities;
- Instream sand and gravel and other materials mining operations in the vicinity of the bridge; and
- Histories of flood damaged highways and bridges.

d. Bridges located on stream reaches with adverse flow characteristics, including:

- Crossings near stream confluences, especially bridge crossings of tributary streams near their confluence with larger streams;
- Crossings on sharp bends in a stream; and
- Locations on alluvial fans.

STEP 2. Prioritize the scour-susceptible bridges and bridges with unknown foundations, by conducting a preliminary office and field examination of the list of structures compiled in step 1, using the following factors as a guide:

a. The potential for bridge collapse or for damage to the bridge in the event of a major flood; and

b. The functional classification of the highway on which the bridge is located, and the effect of a bridge collapse on the safety of the traveling public and on the operation of the overall transportation system for the area or region;

STEP 3. Conduct field and office scour evaluations of the bridges on the prioritized list in step 2 using an interdisciplinary team of hydraulic, geotechnical, and structural engineers:

a. The recommended evaluation procedure is to estimate scour for a superflood, a flood exceeding the 100-year flood, and then analyze the foundations for vertical and lateral stability for this condition of scour. This evaluation approach is the same as the check procedure set forth in Section 3.2, step 8. FHWA recommends using the 500-year flood or a flow 1.7 times the 100-year flood for this purpose where the 500-year flood is unknown. An overtopping flood will be used where applicable. The difference between designing a new bridge and assessing an old bridge is simply that the location and geometry of a new bridge and its foundation are not fixed as they are for an existing bridge. Thus, the same steps for predicting scour at the piers and abutments should be carried out for an existing bridge as for a new bridge. As with the design of a new bridge, engineering judgment must be exercised in establishing the total scour depth for an existing bridge. The maximum scour depths that the existing foundation can withstand are compared with the total scour depth. An engineering assessment must then be made as to whether the bridge should be classified as a scour-critical bridge; that is, whether the bridge foundations cannot withstand the total scour without failing.

b. Enter the results of the scour evaluation study in the bridge inventory in accordance with the instructions in the FHWA "Bridge Recording and Coding Guide" (see Appendix E).[7] Update the list of the scour-critical bridges.

- Bridges assessed as "low risk" for Item 113 (scour-critical bridges) should be coded as an "9, 8, 7, 5, or 4."
- Bridges with unknown foundations (except for interstate bridges) should be coded as a "U" in Item 113, indicating that a scour evaluation/calculation has not been made. It is recommended that only those bridges with unknown foundations which have observed scour, receive scour evaluation prior to the deployment of instrumentation currently being developed to determine foundation type and depth.
- Bridges over tidal waterways (except for interstate bridges) that have not been evaluated for scour, but which are considered low risk should be coded "T". Bridge should be monitored with regular inspection cycle and with appropriate underwater inspections.
- Bridges assessed to be "scour susceptible" are coded as "6" for Item 113 until such time that further scour evaluations determine foundation conditions.
- Interstate bridges with unknown foundations or over tidal waterways should be coded as 6.
- Bridges considered scour-critical based on an evaluation should be coded as a 3 for Item 113.

STEP 4. For bridges identified as scour critical from the office and field review in step 2, determine a plan of action (see Chapter 7) for correcting the scour problem, including:

a. Interim plan of action to protect the public until the bridge can be replaced or scour countermeasures installed. This could include:

- Timely installation of temporary scour countermeasures such as monitoring or riprap and monitoring.
- Plans for monitoring scour-critical, unknown foundation, and tidal bridges during, and inspection after flood events, and for blocking traffic, if needed, until scour countermeasures are installed.
- Immediate bridge replacement or the installation of permanent scour countermeasures depending upon the risk involved.

b. Establishing a time table for step 5 discussed below.

STEP 5. After completing the scour evaluations for the list of potential problems compiled in step 1, the remaining waterway bridges included in the State's bridge inventory should be evaluated. In order to provide a logical sequence for accomplishing the remaining bridge scour evaluations, another bridge list should be established, giving priority status to the following:

a. The functional classification of the highway on which the bridge is located with highest priorities assigned to arterial highways and lowest priorities to local roads and streets.

b. Bridges that serve as vital links in the transportation network and whose failure could adversely affect area or regional traffic operations.

The ultimate objectives of this scour evaluation program are to (1) review all bridges over streams in the National Bridge Inventory; (2) determine those foundations which are stable for estimated scour conditions and those which are not, and (3) provide interim scour protection for scour-critical bridges until adequate scour countermeasures are installed. This may include interim scour protection such as riprap, closing the bridge during high water, monitoring of scour-critical bridges during, and inspection after flood events. The final objective (4) would be to replace the bridge or install scour countermeasures in a timely manner, depending upon the perceived risk involved.

5.3 Conducting Scour Evaluation Studies

An overall plan should be developed for conducting engineering bridge scour evaluation studies. It is recommended that each State develop its own plan for making engineering scour evaluations based on its own particular needs. The FHWA offers the following recommendations in regard to conducting these studies:

1. The first step of the scour evaluation study should be an office review of available information for purposes of assessing the stability of the stream and the adequacy of the

bridge foundations to withstand a superflood (a Q_{500} flood or a flow 1.7 times Q_{100}).

2. The use of worksheets is encouraged since they provide a consistent frame of reference for making field and office reviews and for documenting the results of the investigations.
3. To develop an efficient process for properly evaluating a large number of bridges, a logical sequence needs to be established for conducting the evaluations. This sequence should serve to screen out those bridges where scour is clearly not a problem. For example, sufficient information may be available in the office to indicate that the bridge foundations have been set well below maximum expected scour, and that a field inspection is not necessary for determining that the bridge is not at risk from scour damage. However, a field inspection is generally recommended for bridges over streams that have one or more of the characteristics listed under step 1, of this chapter.

Where adequate hydraulic studies have been prepared and kept for the original bridge design, the scour estimates can be checked or recalculated from this information. Where hydraulic data are not available, it may have to be calculated. For such instances, a "worst-case analysis" is suggested. If the bridge foundations are adequate for worst-case conditions, the bridge can be judged satisfactory. Where the worst-case analysis indicates that a scour problem may exist, further field and office analyses should be made.

5.4 Worst-Case Analysis

The following guide is offered for conducting a worst-case analysis:

5.4.1 Water-Surface Elevations

Information may not be available on the water-surface elevations of the stream at some bridges. This can be compensated for by using procedures developed by the USGS for many states. These procedures provide for estimating depths of flow by using hydrologic area, drainage area, flood frequency, and error of estimate. Using these procedures, a conservative depth-discharge relationship can be determined. This relationship can then be used to develop rough estimates of scour.

5.4.2 Long-Term Aggradation or Degradation

Long-term streambed profile changes will usually be difficult to assess. The main information sources are the records and knowledge of bridge inspectors, maintenance personnel, or others familiar with the bridge site and the behavior of the stream and other streams in the general area. If aggradation or degradation is a problem, there will usually be some knowledge of its occurrence in the area. Cross sections of the stream at the bridge site, for example, when taken by bridge inspectors over a period of time, may indicate a long-term trend in the elevation of the streambed. Field inspections should be made at locations where the streams are known to be active and where significant aggradation or degradation or lateral channel movement is occurring. Further discussion on long-term streambed elevation changes is included in Chapter 2, Chapter 3, and Chapter 4 and HEC-20.[12] Particular attention should be given to bridges at problem sites, as noted earlier in this section. Such bridges should be reviewed in the field. Additional information on conducting field reviews is included in Chapter 6.

5.4.3 Planform Changes

Assessing the significance of planform changes, such as the shifting location of meanders, the formation of islands, and the overall pattern of streams, usually cannot be accomplished in the office. Records and photographs taken by bridge inspectors and maintenance personnel may provide some insight into the nature of the stream for the initial office assessments. Historical aerial photographs of the stream can be extremely valuable in this analysis. Ultimately, an engineering judgment must be made as to whether possible future or existing planform changes represent a hazard to the bridge, and the extent of field work required to evaluate this condition.

5.4.4 Contraction Scour

Contraction scour may be calculated using the equations in Chapter 4 where the amount of overbank and main channel flow is known or can be estimated. The worst-case approach would involve estimating the largest reasonable amount of overbank flow on the floodplain beyond the bridge abutments and then calculating contraction scour on this basis. More detailed analyses are recommended for bridges at problem sites, especially where a large difference in the water-surface elevations may exist up- and downstream of the bridge.

5.4.5 Local Pier Scour

To determine local pier scour use the equations given in Chapter 4.

5.4.6 Local Abutment Scour

Determination of local abutment scour using the procedures and equations in Chapter 4 requires an understanding of flow depths and velocities, and the flow distribution on the floodplain upstream of the bridge. However, some preliminary judgments may be developed as to the expected scour potential through an assessment of the abutment location, the amount of flow in the floodplain beyond the abutment and the extent of protection provided (riprap, guide banks, etc.). It should be noted that the equations given in the literature are based on flume experiments and predict very conservative abutment scour depths.

5.5 Documenting Bridge Scour Assessments

A record should be made of the results of field and office reviews of bridge scour assessments. Item 113, Scour Critical Bridges, of the FHWA document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" requires states to identify the current status of bridges regarding vulnerability to scour.[7]

Go to Chapter 6

Chapter 6 : HEC 18
Inspection of Bridges for Scour

Go to Chapter 7

6.1 Introduction

There are two main objectives to be accomplished in inspecting bridges for scour:

1. To accurately record the present condition of the bridge and the stream; and
2. To identify conditions that are indicative of potential problems with scour and stream stability for further review and evaluation by others.

In order to accomplish these objectives, the inspector needs to recognize and understand the interrelationship between the bridge, the stream, and the floodplain. Typically, a bridge spans the main channel of a stream and perhaps a portion of the floodplain. The road approaches to the bridge are typically on embankments which obstruct flow on the floodplain. This overbank or floodplain flow must, therefore, return to the stream at the bridge and/or overtop the approach roadways. Where overbank flow is forced to return to the main channel at the bridge, zones of turbulence are established and scour is likely to occur at the bridge abutments. Further, piers and abutments may present obstacles to flood flows in the main channel, creating conditions for local scour because of the turbulence around the foundations. After flowing through the bridge, the floodwater will expand back to the floodplain, creating additional zones of turbulence and scour.

The following sections in this chapter present guidance for the bridge inspector's use in developing a comprehension of the overall flood flow patterns at each bridge inspected; and the use of this information for rating the present condition of the bridge and the potential for damage from scour. When an actual or potential scour problem is identified by a bridge inspector, the bridge should be further evaluated by an interdisciplinary team using the approach discussed in Chapter 5. The results of this evaluation should be recorded under Item 113 of the "Bridge Recording and Coding Guide" (see Appendix E).[6]

If the bridge is determined to be scour critical, a plan of action (Chapter 7) should be developed for installing scour countermeasures. In this case, the rating of the bridge substructure (Item 60 of the "Bridge Recording and Coding Guide") should be revised to reflect the effect of the scour on the substructure.[7]

6.2 Office Review

It is desirable to make an office review of bridge plans and previous inspection reports prior to making the bridge inspection. Information obtained from the office review provides a better basis for inspecting the bridge and the stream. Items for consideration in the office review include:

1. Has an engineering scour evaluation study been made? If so, is the bridge scour-critical?
2. If the bridge is scour-critical, has a plan of action been made for monitoring the bridge and/or installing scour countermeasures?
3. What do comparisons of streambed cross sections taken during successive inspections reveal about the streambed? Is it stable? Degrading? Aggrading? Moving laterally? Are there scour holes around piers and abutments?
4. What equipment is needed (rods, poles, sounding lines, sonar, etc.) to obtain streambed cross sections?
5. Are there sketches and aerial photographs to indicate the planform location of the stream and whether the main channel is changing direction at the bridge?
6. What type of bridge foundation was constructed? (Spread footings, piles, drilled shafts, etc.) Do the foundations appear to be vulnerable to scour?
7. Do special conditions exist requiring particular methods and equipment (divers, boats, electronic gear for measuring stream bottom, etc.) for underwater inspections?
8. Are there special items that should be looked at? (Examples might include damaged riprap, stream channel at adverse angle of flow, problems with debris, etc.)

6.3 Bridge Inspection

During the bridge inspection, the condition of the bridge waterway opening, substructure, channel protection, and scour countermeasures should be evaluated, along with the condition of the stream.

The FHWA "Bridge Recording and Coding Guide" (see Appendix E) containsmaterial for the following three items:[7]

1. Item 60: Substructure,
2. Item 61: Channel and Channel Protection, and
3. Item 71: Waterway Adequacy.

The guidance in the "Bridge Recording and Coding Guide" for rating the present condition of Items 61 and 71 is set forth in detail. Guidance for rating the present condition of Item 60, Substructure, is general and does not include specific details for scour. The following sections present approaches to evaluating the present condition of the bridge foundation for scour and the overall scour potential at the bridge.

6.3.1 Assessing The Substructure Condition

Item 60, Substructure, is the key item for rating the bridge foundations for vulnerability to scour damage. When a bridge inspector finds that a scour problem has already occurred, it should be considered in the rating of Item 60. Both existing and potential problems with scour should be reported so that a scour evaluation can be made by an interdisciplinary team. The scour evaluation is reported on Item 113 in the revised "Bridge Recording and Coding Guide."[(7)] If the bridge is determined to be scour critical, the rating of Item 60 should be evaluated to ensure that existing scour problems have been considered. The following items are recommended for consideration in inspecting the present condition of bridge foundations:

1. Evidence of movement of piers and abutments;
   - Rotational movement (check with plumb line),
   - Settlement (check lines of substructure and superstructure, bridge rail, etc., for discontinuities; check for structural cracking or spalling), and
   - Check bridge seats for excessive movement.

2. Damage to scour countermeasures protecting the foundations (riprap, guide banks, sheet piling, sills, etc.),

3. Changes in streambed elevation at foundations (undermining of footings, exposure of piles), and

4. Changes in streambed cross section at the bridge, including location and depth of scour holes.

In order to evaluate the conditions of the foundations, the inspector should take cross sections of the stream, noting location and condition of streambanks. Careful measurements should be made of scour holes at piers and abutments, probing soft material in scour holes to determine the location of a firm bottom. If equipment or conditions do not permit measurement of the stream bottom, this condition should be noted for further action.

6.3.2 Assessing Scour Potential at Bridges

The items listed in Table 15 are provided for bridge inspectors' consideration in assessing the adequacy of the bridge to resist scour. In making this assessment, inspectors need to understand and recognize the interrelationships between Item 60 (Substructure), Item 61 (Channel and Channel Protection), and Item 71 (Waterway Adequacy). As noted earlier, additional follow-up by an interdisciplinary team should be made utilizing Item 113 (Scour Critical Bridges) when the bridge inspection reveals a potential problem with scour (see Appendix E).

6.3.3 Underwater Inspections

Perhaps the single most important aspect of inspecting the bridge for actual or potential damage from scour is the taking and plotting of measurements of stream bottom elevations in relation to the bridge foundations. Where conditions are such that the stream bottom cannot be accurately measured by rods, poles, sounding lines or other means, other arrangements need to be made to determine the condition of the foundations. Other approaches to determining the cross section of the streambed at the bridge include:

1. Use of divers; and
2. Use of electronic scour detection equipment (Appendix F).

For the purpose of evaluating resistance to scour of the substructure under Item 60 of the "Bridge Recording and Coding Guide," the questions remain essentially the same for foundations in deep water as for foundations in shallow water:[7]

1. How does the stream cross section look at the bridge?
2. Have there been any changes as compared to previous cross section measurements? If so, does this indicate that (1) the stream is aggrading or degrading; or (2) local or contraction scour is occurring around piers and abutments?
3. What are the shapes and depths of scour holes?
4. Is the foundation footing, pile cap, or the piling exposed to the streamflow; and if so, what is the extent and probable consequences of this condition?
5. Has riprap around a pier been moved or removed?

6.3.4 Notification Procedures

A positive means of promptly communicating inspection findings to proper agency personnel must be established. **Any condition that a bridge inspector considers to be of an emergency or potentially hazardous nature should be reported immediately**. That information as well as other conditions which do not pose an immediate hazard, but still warrant further action, should be conveyed to the interdisciplinary team for review.

Click here to view Table 15. Assessing the Scour Potential at Bridges.

1. Upstream Conditions
2. Conditions at Bridge
3. Downstream Conditions

A report form is, therefore, needed to communicate pertinent problem information to the hydraulic/ geotechnical engineers. An existing report form may currently be used by bridge inspectors within a State highway agency to advise maintenance personnel of specific needs. Regardless of whether an existing report is used or a new one is developed, a bridge inspector should be provided the means of advising the interdisciplinary team of problems in a timely manner.

Go to Chapter 7

Chapter 7 : HEC 18

Plan of Action for Installing Scour Countermeasures

Go to Appendix A

7.1 Introduction

A plan of action has three primary components:

1. Timely installation of temporary scour countermeasures (e.g., riprap).
2. Development and implementation of a monitoring program.
3. Schedule for timely design and construction of permanent scour countermeasures.

Scour countermeasures are features generally incorporated after the initial construction of a bridge to make it less vulnerable to damage or failure from scour.

7.1.1 New Bridges

For new bridges, recommended scour countermeasures have been addressed in Chapter 3 and Chapter 4. In summary, the best solutions for minimizing scour damage include:

1. Locating the bridge to avoid adverse flood flow patterns,
2. Streamlining bridge elements to minimize obstructions to the flow,
3. Design foundations safe from scour,
4. Founding bridge pier foundations sufficiently deep to not require riprap or other countermeasures, and
5. Founding abutment foundations above the estimated local scour depth when the abutment is protected by well designed riprap or other suitable countermeasures.

7.1.2 Existing Bridges

For existing bridges, the alternatives available for protecting the bridge from scour are listed below in a rough order of cost:

1. Monitoring scour depths and closing bridge if excessive,
2. Providing riprap at piers and monitoring,
3. Providing riprap at abutments and monitoring,
4. Constructing guide banks (spur dikes),
5. Constructing channel improvements,
6. Strengthening the bridge foundations,

7. Constructing sills or drop structures, and
8. Constructing relief bridges or lengthening existing bridges.

These alternatives should be evaluated using sound hydraulic engineering practice.

In developing a plan of action for protecting an existing scour-critical bridge, the four aspects that need to be considered are:

1. Monitoring, inspecting, and possibly closing a bridge until the countermeasures are installed,
2. Installing temporary scour countermeasures, such as riprap around a pier, along with monitoring a bridge during high flow,
3. Selecting and designing scour countermeasures, and
4. Scheduling construction of scour countermeasures.

These considerations are discussed in the following sections.

7.2 Monitoring, Inspecting, and Closing Scour-Critical Bridges

As noted in Chapter 5, special attention should be given to monitoring scour-critical bridges during and after flood events. The plan-of-action for a bridge should include special instructions to the bridge inspector, including guidance as to when a bridge should be closed to traffic. Guidance should be given to other DOT officials on bridge closure. The intensity of the monitoring effort is related to the risk of scour hazard, as determined from the scour evaluation study. The following items are recommended for consideration when developing the plan-of-action monitoring effort.

1. Information on any existing rotational movement of abutments and piers or settlement of foundations.
2. Information on rates of streambed degradation, aggradation, or lateral movement based on analysis of changes in stream cross sections taken during successive bridge inspections, sketches of the stream platform, aerial photographs, etc.
3. Recommended procedures and equipment for taking measurements of streambed elevations (use of rods, probes, weights, portable sonic equipment, etc.) during and after floods.
4. Guidance on maximum permissible scour depths, flood flows, water surface elevations, etc., beyond which the bridge should be closed to traffic.
5. Reporting procedures for handling excess scour, larger than normal velocities and water surface elevation or discharge that may warrant bridge closure. Develop a chain of command with authority to close bridges.
6. Instructions regarding the checking of streambed levels in deep channels where accurate measurements cannot be made from the bridge (use of divers, electronic instruments such as sonar, radar, etc.).

7. Instructions for inspecting existing countermeasures such as riprap, dikes, sills, etc.
8. Forms and procedures for documenting inspection results and instructions regarding follow-up actions when necessary.
9. Installation of scour depth warning devices, such as low cost sonar sonic fathometers and mechanical devices (see Appendix F).[78,79]
10. Instructions for checking the operation of fixed monitoring instruments.

7.3 Temporary Countermeasures

Monitoring of bridges during high flow may indicate that collapse from scour is imminent. It may not be advantageous, however, to close the bridge during high flow because of traffic volume, poor alternate routes, the need for emergency vehicles to use the bridge, etc. Temporary scour countermeasures such as riprap or fixed monitoring instruments could be installed, allaying the need for immediate closure. Temporary countermeasure installed at a bridge combined with provisions for monitoring during and inspection after high flows could provide for the safety of the public without closing the bridge.

7.4 Scheduling Construction of Scour Countermeasures

The engineering scour evaluation study should address the risk of failure at scour-critical bridges so that priorities and schedules can be prepared for installation of scour countermeasures at differing bridge sites. In some cases, the risk may be obvious, as where an inspection reveals that a spread footing for a pier has been partially undermined. Immediate action is warranted. In other cases, the need for immediate action is not so apparent, and considerable judgment must be exercised. An example of the latter case is where a stream meander is gradually encroaching upon a bridge abutment. A judgment must be made on the risk associated with the rate of change of the meander and its probable effect on the abutment and associated foundation.

Gradual river changes are common. As a consequence, the engineer may wait too long to take action. As the degree of encroachment and scour hazard increases, the number of alternative countermeasures available is decreased and costs of correction are corresponding increased. In addition, monitoring a bridge during high flows and inspection after high flow may not determine that a bridge is about to collapse from scour unless a time history of scour can be obtained.

7.5 Types of Countermeasures

An overview of commonly used scour countermeasures is provided below, along with references for obtaining design procedures and criteria for their application to a specific site. Selection of the appropriate countermeasure is best accomplished through a field and office evaluation of the conditions at the stream crossing (see also, HEC-20 Chapter 5 and Chapter 6).[12]

7.5.1 Rock Riprap at Piers and Abutments

The FHWA continues to evaluate how best to design rock riprap at bridge piers and abutments.

Present knowledge is based on research conducted under laboratory conditions with little field verification, particularly for piers. Flow turbulence and velocities around a pier are of sufficient magnitude that large rocks move over time. Bridges have been lost (Schoharie Creek bridge for example) due to the removal of riprap at piers resulting from turbulence and high velocity flow. Usually this does not happen during one storm, but is the result of the cumulative effect of a sequence of high flows. **Therefore, if rock riprap is placed as scour protection around a pier, the bridge should be monitored and inspected during and after each high flow event to insure that the riprap is stable**.

Sizing Rock Riprap at Abutments. The FHWA conducted two research studies in a hydraulic flume to determine equations for sizing rock riprap for protecting abutments from scour.[80,81] One study investigated vertical wall and spill-through abutments which encroached 28 and 56 percent on the floodplain, respectively.[80] The second study investigated spill-through abutment which encroached on a floodplain with an adjacent main channel (see Figure 27). Encroachment varied from the largest encroachment used in the first study to a full encroachment to the edge of main channel bank. For spill-through abutments in both studies, the rock riprap consistently failed at the toe downstream of the abutment centerline (see Figure 28). For vertical wall abutments, the first study consistently indicated failure of the rock riprap at the toe upstream of the centerline of the abutment.

Field observations and laboratory studies reported in HIRE indicate that with large overbank flow or large drawdown through a bridge opening that scour holes develop on the side slopes of spill-through abutments and the scour can be at the upstream corner of the abutment.[13] In addition, flow separation can occur at the downstream side of a bridge (either with vertical wall or spill-through abutments). This flow separation causes vertical vortices which erode the approach embankment and the downstream corner of the abutment.

For Froude Numbers $V/(gy)^{1/2} \leq 0.80$, the recommended design equation for sizing rock riprap for spill-through and vertical wall abutments is in the form of the Isbash relationship:

Equation 8-1 (81)

where:

D_{50} = Median stone diameter, m
V = Characteristic average velocity in the contracted section (explained below), m/s
S_s = Specific gravity of rock riprap
g = Gravitational acceleration, 9.81 m/s^2
y = Depth of flow in the contracted bridge opening, m
K = 0.89 for a spill-through abutment
1.02 for a vertical wall abutment

For Froude Numbers >0.80, Equation 82 is recommended:[82]

Equation 8-2 (82)

where:

K = 0.61 for spill-through abutments
= 0.69 for vertical wall abutments

In both equations, the coefficient K, is a velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure. Both of these equations are envelop relationships that were forced to overpredict 90 percent of the laboratory data.[80,81,82]

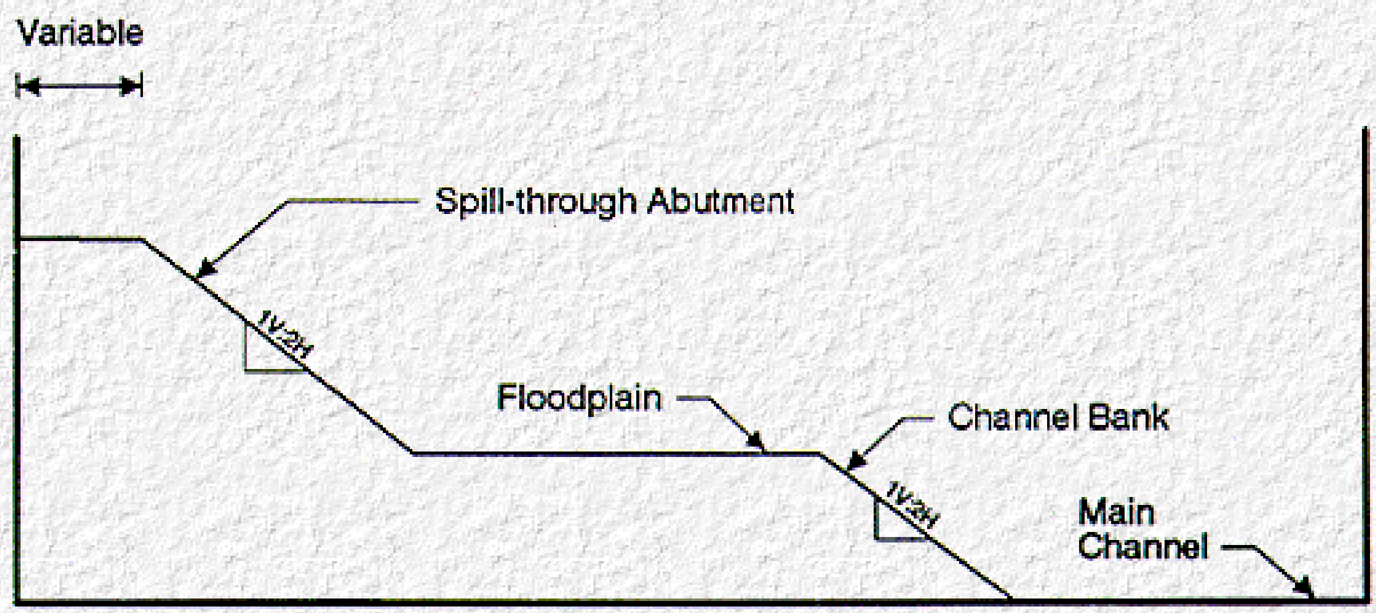

Figure 27. Section View of a Typical Setup of Spill-through Abutment on a Floodplain with Adjacent Main Channel [81]

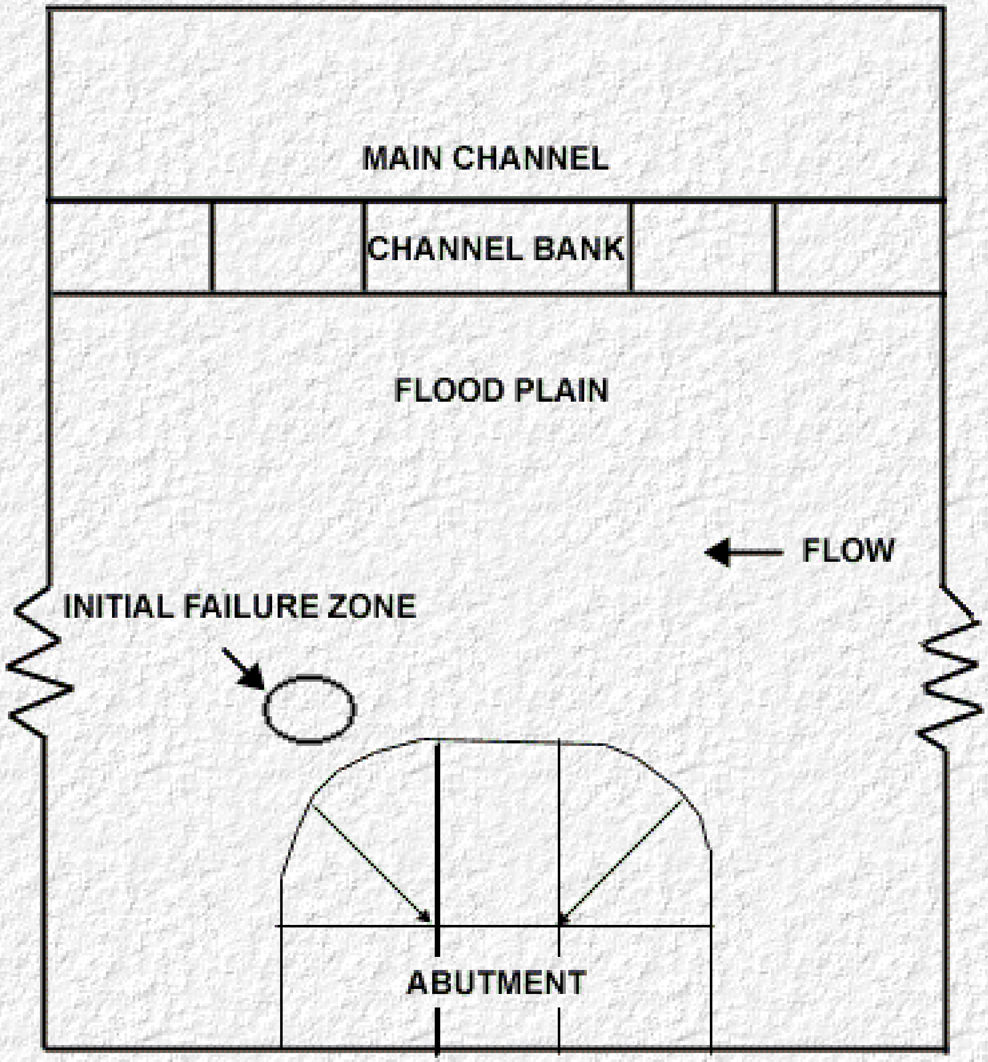

Figure 28. Plan View of the Location of Initial Failure Zone of Rock Riprap for Spill-Through Abutment (80, 81)

A recommended procedure for selecting the characteristic average velocity is as follows:

1. Determine the set-back ratio (SBR) of each abutment. SBR is the ratio of the set-back length to channel flow depth. The set-back length is the distance from the near edge of the main channel to the toe of abutment.

SBR = Set-back length/average channel flow depth

> a. If SBR is less than 5 for both abutments, compute a characteristic average velocity, Q/A, based on the entire contracted area through the bridge opening. This includes the total upstream flow, exclusive of that which overtops the roadway. The WSPRO average velocity through the bridge opening is also appropriate for this step.
>
> b. If SBR is greater than 5 for an abutment, compute a characteristic average velocity, Q/A, for the respective overbank flow only. Assume

that the entire respective overbank flow stays in the overbank section through the bridge opening. This velocity can be approximated by a hand calculation using the cumulative flow areas in the overbank section from WSPRO, or from a special WSPRO run using an imaginary wall along the bank line.

c. If SBR for an abutment is less than 5 and SBR for the other abutment at the same site is more than 5, a characteristic average velocity determined from Step 1a for the abutment with SBR less than 5 may be unrealistically low. This would, of course, depend upon the opposite overbank discharge as well as how far the other abutment is set back. For this case, the characteristic average velocity for the abutment with SBR less than 5 should be based on the flow area limited by the boundary of that abutment and an imaginary wall located on the opposite channel bank. The appropriate discharge is bounded by this imaginary wall and the outer edge of the floodplain associated with that abutment.

2. Compute rock riprap size from Equation 81and Equation 82, based on the Froude Number limitation for these equations.

3. Determine extent of rock riprap.

a. The apron at the toe of the abutment slope should extend along the entire length of the abutment toe, around the curved portions of the abutment to the point of tangency with the plane of the embankment slopes.

b. The apron should extend from the toe of the abutment into the bridge waterway a distance equal to twice the flow depth in the overbank area near the embankment, but need not exceed 7.5 m (see Figure 29).[83]

c. Spill-through abutment slopes should be protected with rock riprap size computed from Equation 81and Equation 82 to and elevation 0.15 m above expected high water elevation for the design flood. Upstream and downstream coverage should agree with step 3a except that the downstream riprap should extend back from the abutment 2 flow depths or 7.5 m which ever is larger to protect the approach embankment. Several states in the southeast use a guide bank 15 m long at the downstream end of the abutment to protect the downstream side of the abutment.

d. The rock riprap thickness should not be less than the larger of either 1.5 times D_{50} or D_{100}. The rock riprap thickness should be increased by 50 percent when it is placed under water to provide for the uncertainties associated with this type of placement.

e. The rock riprap gradation and the potential need for underlying filter material must be considered.

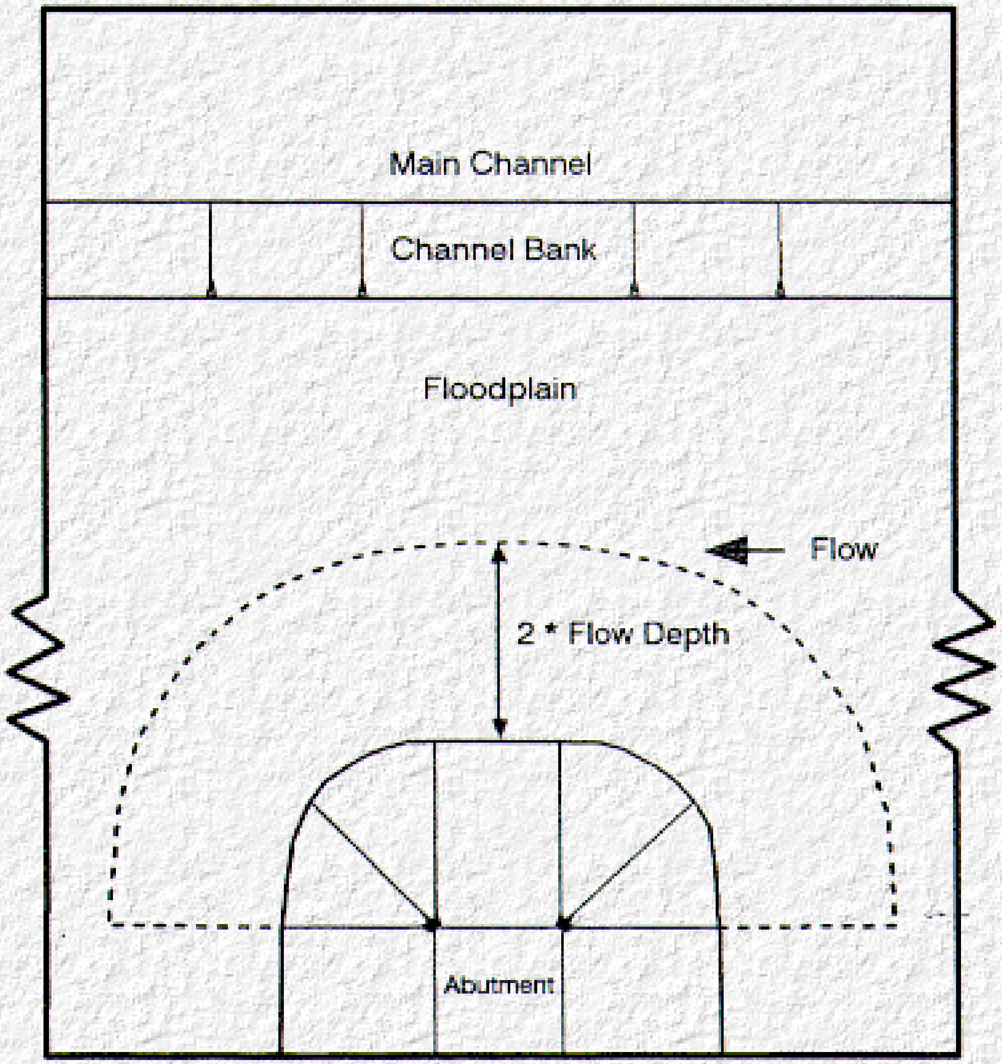

Figure 29. Plan View of the Extension of Rock Riprap Apron [83]

Sizing Riprap at Piers. **Riprap is not a permanent countermeasure for scour at piers for existing bridges and not to be used for new bridges.** Determine the D_{50} size of the riprap using the rearranged Isbash equation (see HIRE) to solve for stone diameter (in meters, for fresh water):[13]

$$D_{50} = \frac{0.69(KV)^2}{(S_s - 1)2g} \qquad (83)$$

where:

D_{50} = Median stone diameter, m
K = Coefficient for pier shape
V = Velocity on pier, m/s
S_s = Specific gravity of riprap (normally 2.65)
g = 9.81 m/s^2
K = 1.5 for round-nose pier
K = 1.7 for rectangular pier

To determine V multiply the average channel velocity (Q/A) by a coefficient that ranges from 0.9 for a pier near the bank in a straight uniform reach of the stream to 1.7 for a pier in the main current of flow around a bend.

1. Provide a riprap mat width which extends horizontally at least two times the pier width, measured from the pier face.

2. Place the top of a riprap mat at the same elevation as the streambed. The deeper the riprap is placed into the streambed, the less likely it will be moved. Placing the bottom of a riprap mat on top of the streambed is discouraged. In all cases where riprap is used for scour control, the bridge must be monitored during and inspected after high flows.

It is important to note that it is a disadvantage to bury riprap so that the top of the mat is below the streambed because inspectors have difficulty determining if some or all of the riprap has been removed. Therefore, it is recommended to place the top of a riprap mat at the same elevation as the streambed.

. The thickness of the riprap mat should be three stone diameters (D_{50}) or more.
b. In some conditions, place the riprap on a geotextile or a gravel filter. However, if a well-graded riprap is used, a filter may not be needed. In some flow conditions it may not be possible to place a filter or if the riprap is buried in the bed a filter may not be needed.
c. The maximum size rock should be no greater than twice the D_{50} size.

7.5.2 Guide Banks

Methods for designing guide banks are contained in the FHWA publication Hydraulic Design Series No. 1, "Hydraulics of Bridge Waterways" and HEC-20.(84,12) The hydraulic effect of guide banks can be modeled through the use of the FHWA software WSPRO.(24) The purpose of the guide bank is to provide a smooth transition for flows on the floodplain returning to the main channel at the bridge. The guide bank also moves the point of maximum scour upstream, away from the abutment and align flows through the bridge opening. Guide banks should be considered for protecting bridge abutments whenever there is a significant amount of flow on the floodplain that must return to the main channel at the bridge.

7.5.3 Channel Improvements

A wide variety of countermeasures are available for stabilizing and controlling flow patterns in streams.

a. Countermeasures for aggrading streams include:

- Contracting the waterway upstream and through the bridge to cause it to scour,
- Construction of upstream dams to create sedimentation basins,
- Periodic cleaning of the channel, and
- Raising the grade of the bridge and approaches.

b. Countermeasures for degrading streams include the construction of sills and the strengthening of foundations as discussed in Section 7.5.5.[12,13]

c. Countermeasures for controlling lateral movement of a stream due to stream meanders include placement of dikes or jetties along the streambanks to redirect the flow through the bridge along a favorable path that minimizes the angle of attack of the current on the bridge foundations. HEC-20 addresses this type of countermeasure in detail.[12] Another useful reference is Transportation Research Board Record 950.[44]

7.5.4 Structural Scour Countermeasures

The use of structural designs to underpin existing foundations is discussed in the AASHTO Manual for Bridge Maintenance.[85] While structural measures may be more costly, they generally provide more positive protection against scour than countermeasures such as riprap.

7.5.5 Constructing Sills or Drop Structures

The use of sills and drop structures at bridges to stabilize the streambed and counteract the affects of degradation is discussed in FHWA publications.[12,13]

7.5.6 Constructing Relief Bridges or Extra Spans on the Main Bridge

Providing additional waterway to relieve existing flow conditions is essentially a design problem and the guidance in Chapter 3 and Chapter 4 is applicable to implementation. In some locations with very unstable banks, additional spans may be more cost effective than attempting to stabilize the channel banks in the vicinity of the bridge.

7.6 Summary

The foregoing discussion of countermeasures presents a variety of concepts and approaches for addressing scour problems at bridges. The interdisciplinary scour evaluation team needs to collect and evaluate information about the behavior of streams and flood flow patterns through bridges so that the most appropriate countermeasures are selected for the particular set of site conditions under study. The FHWA publication "Countermeasures for Hydraulic Problems at Bridges (Volume 2, Case Histories)," is recommended as a guide for reviewing the performance of the countermeasures discussed above.[2] This document is summarized in Chapter 5 of HEC-20.[12]

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Appendix A : HEC 18

Use of the Metric System

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The following information is summarized from the Federal Highway Administration, National Highway Institute (NHI) Course No. 12301, "Metric (SI) Training for Highway Agencies." For additional information, refer to the Participant Notebook for NHI Course No. 12301.

In SI there are seven base units, many derived units and two supplemental units (Table A-1). Base units uniquely describe a property requiring measurement. One of the most common units in civil engineering is length, with a base unit of meters in SI. Decimal multiples of meter include the kilometer (1000 m), the centimeter (1 m/100) and the millimeter (1 m/1000). The second base unit relevant to highway applications is the kilogram, a measure of mass which is the inertial of an object. There is a subtle difference between mass and weight. In SI, mass is a base unit, while weight is a derived quantity related to mass and the acceleration of gravity, sometimes referred to as the force of gravity. In SI the unit of mass is the kilogram and the unit of weight/force is the newton. Table A-2 illustrates the relationship of mass and weight. The unit of time is the same in SI as in the English system (seconds). The measurement of temperature is Centigrade. The following equation converts Fahrenheit temperatures to Centigrade, °C = 5/9 × (°F - 32°).

Derived units are formed by combining base units to express other characteristics. Common derived units in highway drainage engineering include area, volume, velocity, and density. Some derived units have special names (Table A-3).

Table A-4 provides useful conversion factors from English to SI units. The symbols used in this table for metric units, including the use of upper and lower case (e.g., kilometer is "km" and a newton is "N") are the standards that should be followed. Table A-5 provides the standard SI prefixes and their definitions.

Table A-6 provides physical properties of water at atmospheric pressure in SI system of units. Table A-7 gives the sediment grade scale and Table A-8 gives some common equivalent hydraulic units.

Click on the hyperlinks below to view the following tables:

- Table A-1. Overview of SI Units
- Table A-2. Relationship of Mass and Weight
- Table A-3. Derived Units With Special Names
- Table A-4. Useful Conversion Factors
- Table A-5. Prefixes

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Appendix B : HEC 18

Interim Procedure For Pressure Flow Scour

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B-1 Abstract

Bridges that become inundated during floods have slightly pressurized flow that impacts bridge piers and creates an aggravated scour condition. Results of two laboratory studies are presented in this paper.

B-2 Introduction

There are approximately 496,000 bridges in the National Bridge Inventory that are over waterways. Approximately one fourth of these bridges have been determined to be scour susceptible and are scheduled for scour evaluations by 1997.

Because of the high risks associated with bridge foundation failures, it behooves bridge owners to include the very large floods in the scour evaluations. While it may be economically prudent to design a roadway or a bridge to overtop at a 25 or a 50-yr. recurrence interval flood, it is probably never economically beneficial to expect the bridge foundation to fail at those frequencies.

Therein lies the problem with pressure-flow scour. The thousands of bridges that are to be evaluated over the next several years will involve many situations with the bridge deck inundated where the flow attacking bridge piers will be under pressure. Most of the research on bridge scour has been conducted under relatively ideal free-surface flow conditions and the prediction equations are based on using representative approach flow depths and velocities.

B-3 Pressure Flow Research Studies

There have been no comprehensive research studies to address this very crucial problem, but there have been two limited laboratory studies that have provided some interim guidelines that could be incorporated into the Federal Highway (FHWA) Hydraulic Engineering Circular on bridge scour. (Richardson, et. al. 1993).

The first study on pressure-flow scour was conducted at Colorado State University by Abed

(1991). Abed tested 10 free-surface pier scour conditions and 15 pressure-flow scour conditions with a submerged model bridge deck in conjunction with a bridge pier. The CSU study kept the sediment size constant at approximately 3.0 mm, the bridge deck and pier size constant, and the depth of submergence of the bridge deck constant at 0.11 m. The variables were the approach flow depth, which ranged from 0.24 m to 0.61 m, and the approach velocity which ranged from 0.305 m/sec to 0.915 m/sec. Most of the experiments were clear-water scour in that there was no sediment transport upstream of the bridge.

The second study was conducted at the FHWA hydraulics lab at the Turner Fairbank Highway Research Center in McLean, VA. That study was an extension of Abed's study and included a series of tests for bridge decks with and without piers to systematically isolate the bridge deck scour from the pier scour. Three bed material sizes (0.43 mm, 1.0 mm and 2.8 mm) were tested. The submergence of the bridge deck varied from slight submergence of the "low steel" beams to complete overtopping of the bridge deck. The flow depth has been kept constant at approximately 0.305 m and the approach velocity varied from 0.42 to approximately 1.0 times the incipient motion velocity for the bed material being tested.

Unfortunately the FHWA flume does not have facilities to recirculate the bed material. The logic for experimenting in the incipient motion velocity range was that it represents near maximum scour conditions and we were looking for relative effects for the pressure flow. The lower velocity results did not seem very meaningful for pressure flow situations because most bridges will have live-bed scour by the time they become submerged, but we did analyze the data in the clear-water range.

Figure 1 and Figure 2 illustrate the test condition for the two studies.

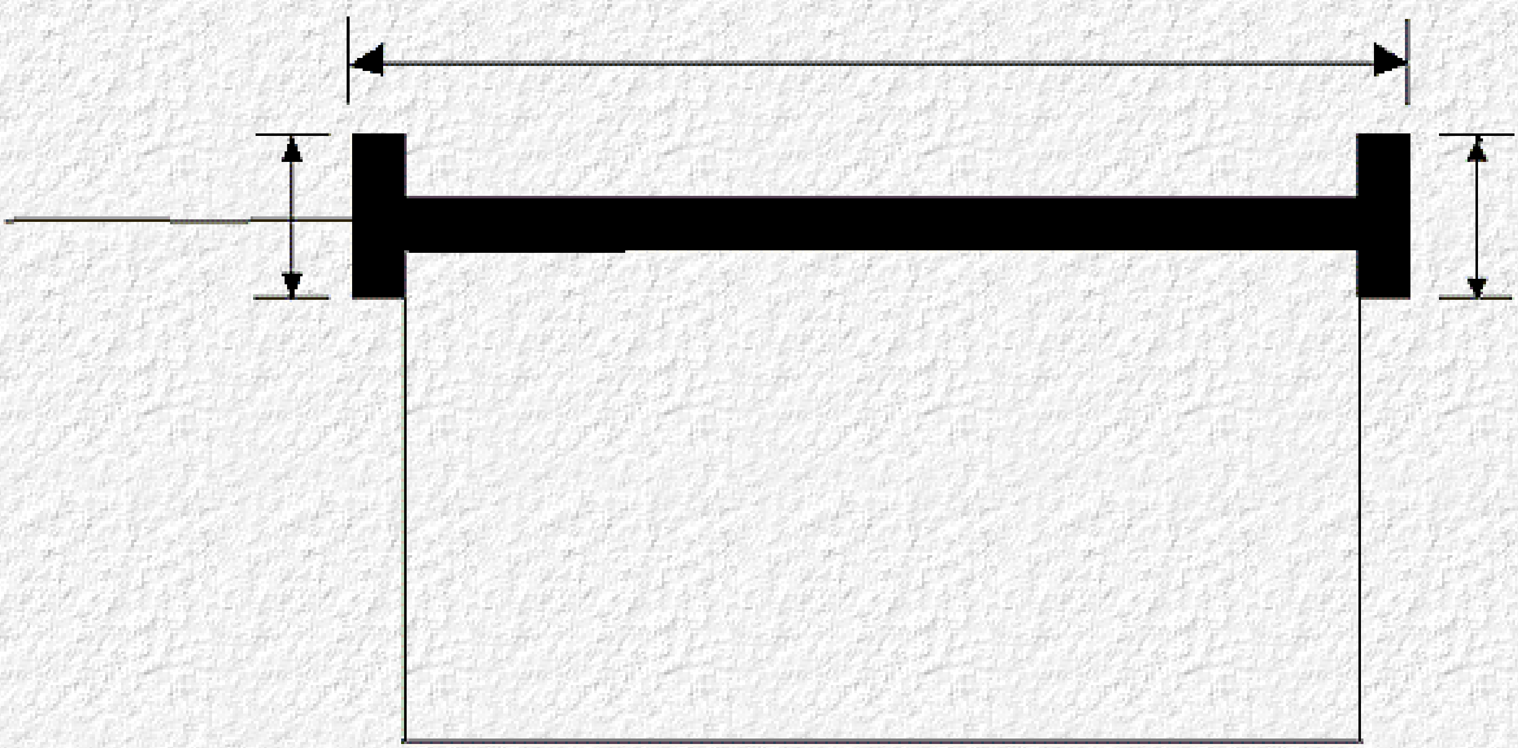


Figure 1. Colorado State University Test Configuration (from Abed 1991)

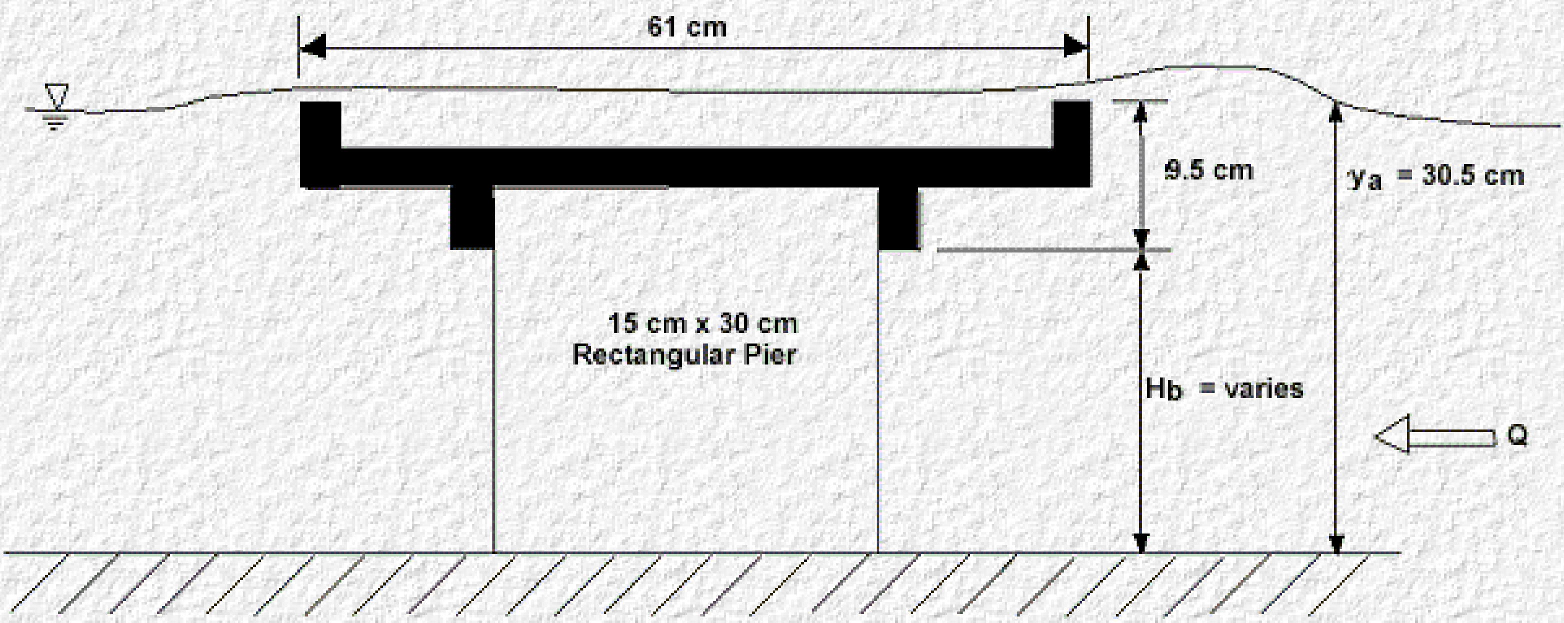


Figure 2. Federal Highway Administration Test Configuration

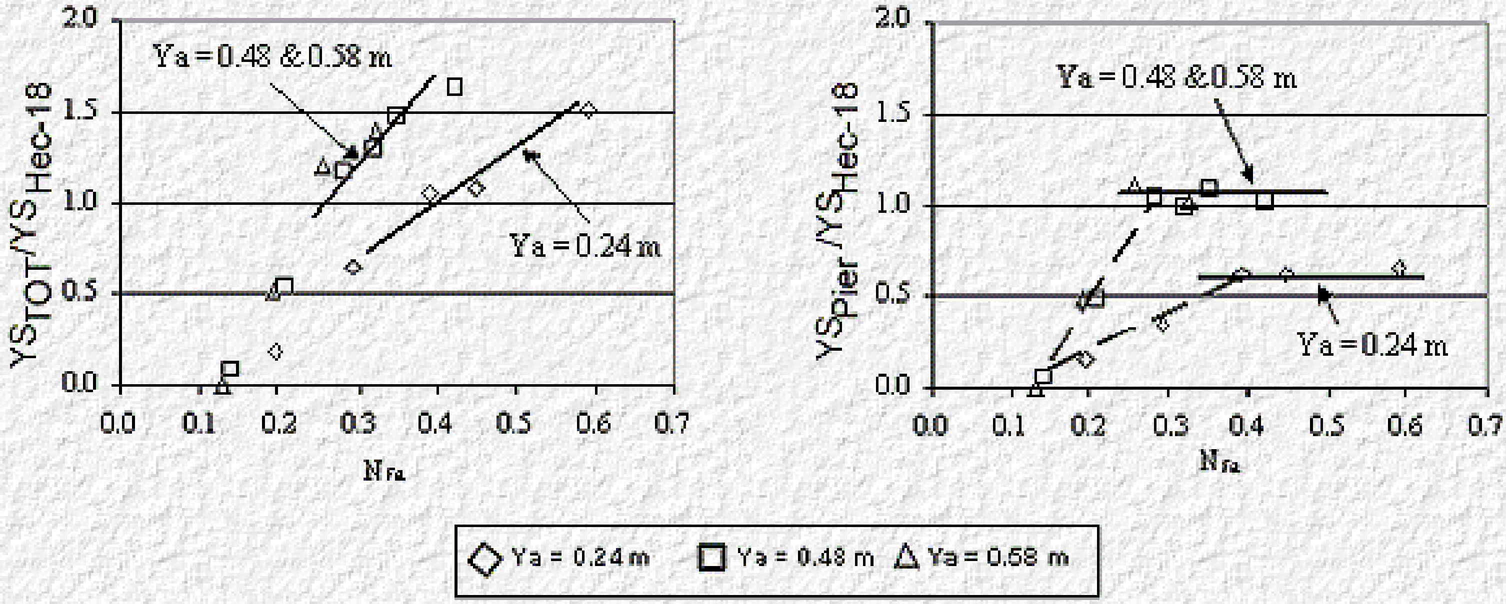
B-4 Results of CSU Study

The CSU results are tabulated in Abed's dissertation. Since the deck scour was not measured directly, the pier scour component could not be isolated accurately, but the apparent pier scour could be approximated by assuming that the scour depth measured midway between the pier and the flume wall was representative of the average deck scour.

When the total pressure-flow scour was compared to the free-surface pier scour with the same approach flow conditions, it was observed that pressure-flow scour was from 2.3 to 10 times the free surface pier scour. One of the first implications from this study was that pier scour could be magnified by 200 to 300 percent when pressure flow occurs.

A gross correction for the combined pressure-flow scour components can be obtained by analyzing the ratios of the total measured pressure flow scour to the estimated HEC 18 pier scour as illustrated in Figure 3a. Based on the observation that ratio was generally less than 1.6 for a range of Froude number from 0.1 to 0.59, an interim guideline for pressure flow scour was incorporated into version 2 of HEC 18 (as revised in April 1993) for that range of Froude number.

Figure 3b is a plot of the apparent pressure-flow pier scour component normalized by the HEC 18 predicted pier scour. The fact that none of those points plotted much above 1.0 was the basis for pursuing the FHWA study aimed especially at isolating the components of pressure flow scour.



a: Total Pressure Flow Scour b. Pressure flow pier scour

Figure 3. Pressure Flow Pier Scour, CSU Data

B-5 Results of FHWA Study

The technique of using pier scour multipliers to account for the pressure flow phenomena does not seem logical since the multiplier would be associated with the particular degree of submergence used in the experiments. Furthermore, the Froude number criteria are associated with the bed material size used in the experiments.

In the FHWA study, three conditions were tested for each approach flow condition. Free- surface pier scour was measured without the bridge deck; bridge deck scour was measured without the pier; and total scour was measured at the pier with both the pier and the deck obstructing the flow. The outstanding observation in the results of this study is that the pressure-flow pier scour component was close to the free-surface pier scour measurements for the same approach flow conditions as illustrated in Figure 4.

The pier scour ratios, near unity, were obtained without adjusting for the increased initial velocity in the opening. That can be attributed to the tendency for the contracted velocity to approach the incipient motion velocity, V_c, as the opening enlarges until particles will no longer move out. The approach velocities used for the free-surface pier scour measurements were close to V_c for the FHWA experiments; thus the close correlation between the pressure flow pier scour components and the free-surface pier scour measurements is reasonable.

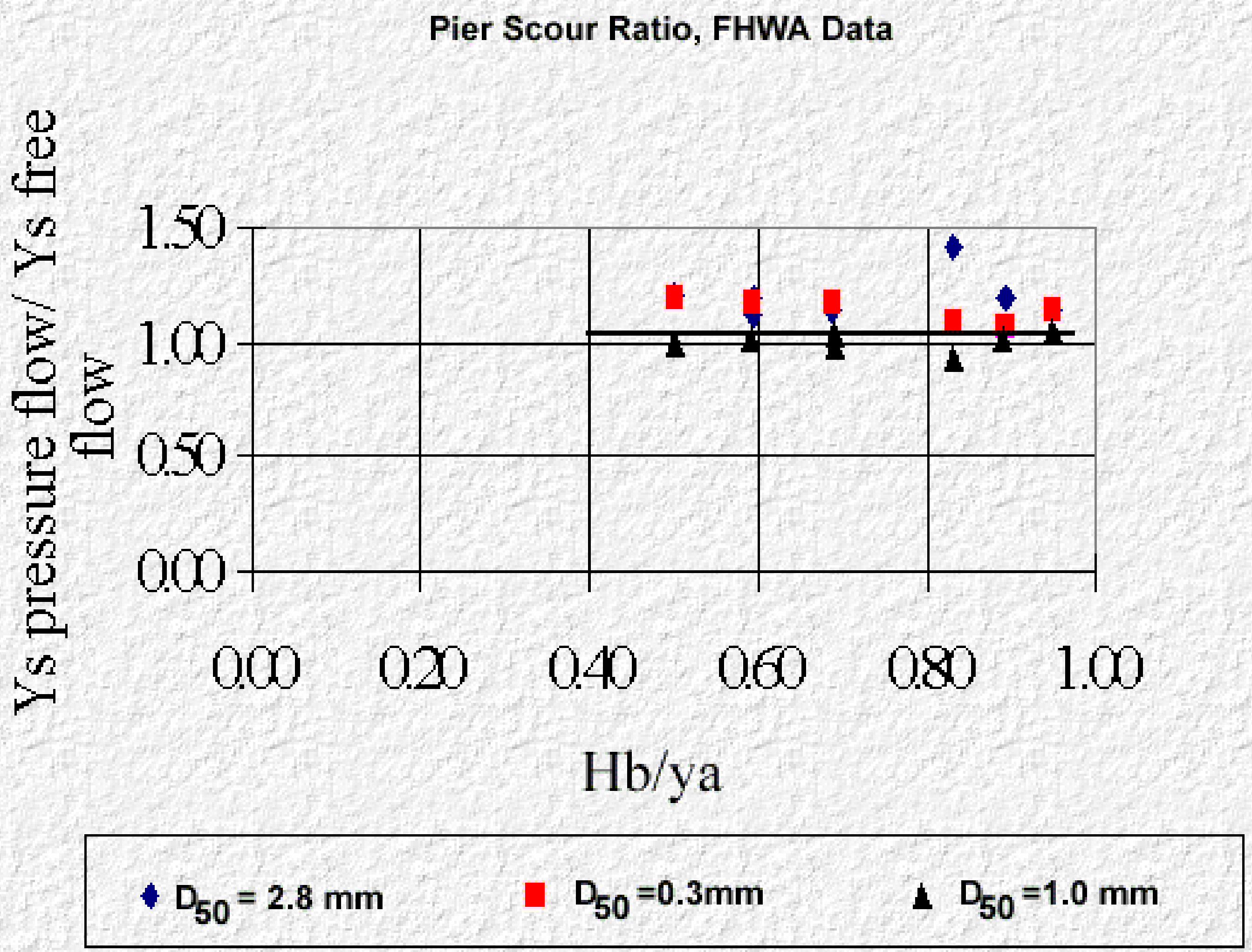


Figure 4. Pressure Flow Pier Scour Ratio, FHWA Data

The deck scour component is a form of vertical contraction scour. A simplified procedure for estimating this component is to assume that the bridge opening enlarges until the velocity in the opening equals the incipient motion velocity, V_c. The velocity in the bridge opening is based on the discharge through the opening which is the total discharge less the discharge that overtops the roadway.

Figure 5 shows that this simplified procedure tends to underestimate the laboratory bridge deck scour measurements. The under predictions can be attributed to a distorted vertical velocity distribution associated with the diving currents from the bridge deck. The velocities near the bed tend to be higher than they would be for a fully developed velocity distribution with the same average velocity. When we first presented this paper at the 1993 ASCE Hydraulics Conference (Jones, et al 1993) we claimed that the simplified procedure could be forced to overpredict all but two data points by increasing the assumed discharge through the bridge opening by 10 percent, but the experiments were not complete at that time and we had not adjusted the absolute scour measurements for ultimate depths that would have occurred if we had run longer duration tests. After we analyzed our data more closely, we realized that the 10 percent adjustment would not be adequate and we needed a more sophisticated procedure.

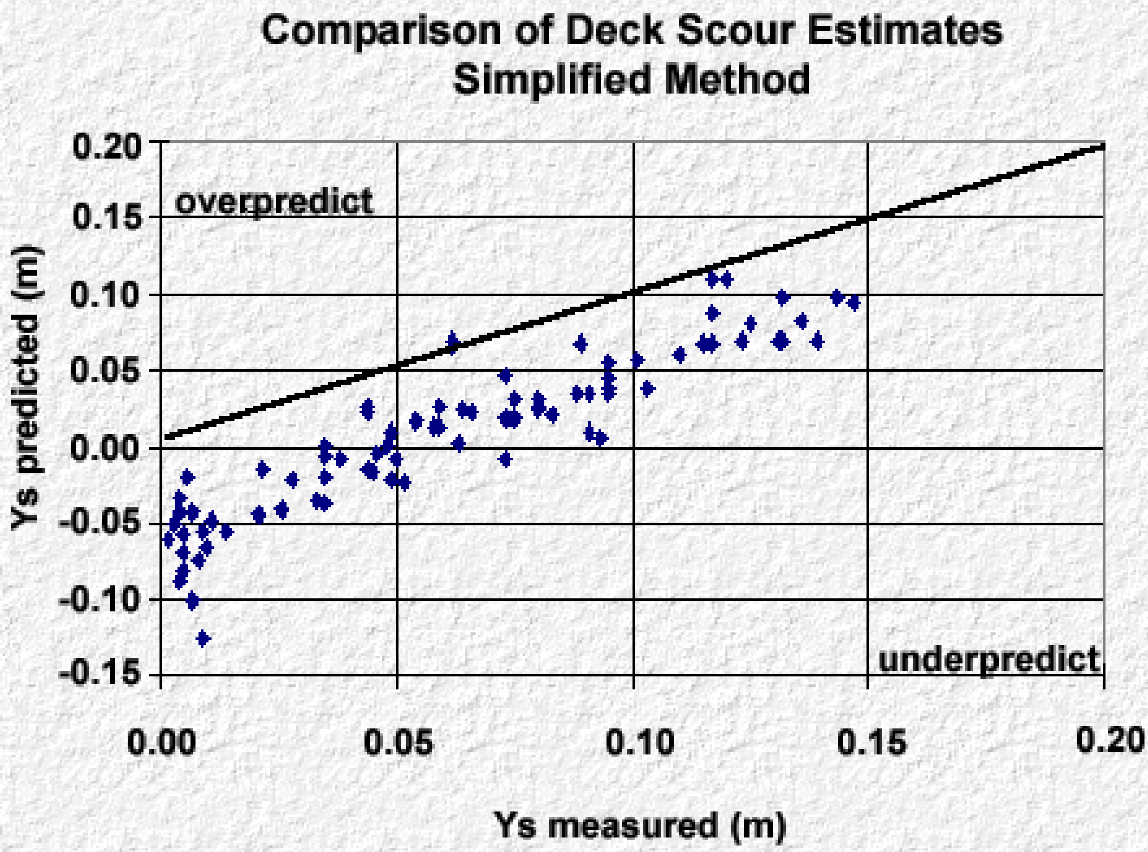


Figure 5. Comparison of Deck Scour Estimates, Simplified Procedure

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B-6 Deck Scour Results

Figure 6 is a definition sketch for the deck scour component. The simplified procedure with the 10 percent correction would suggest:

$$H_b + y_s = 1.10\ q_{br}/V_c$$

where: q_{br} = the unit discharge that passes under the bridge deck.

Since that procedure did not prove to be adequate, we reasoned that the parameters that should affect the adjustment factor were the Froude number, the vertical contraction ratio defined as $H_b/(y_a - w)$, and V_a/V_c. Umbrell et al (1995) analyzed the clear-water data and determined that the best fit to the data was the following relationship:

$$H_o + \frac{y_s}{y_a} = 1.1021\left[\left(1 - \frac{w}{y_a}\right)\frac{V_a}{V_c}\right]^{0.6031}$$

This equation was based on 81 observations and had a correlation coefficient R_2 = 0.81. This equation is an improvement over the simplified procedure, but it is limited in application to the clear- water zone where $V_a < V_c$. It will surely give unreasonable results if V_a is much greater than

V_c.

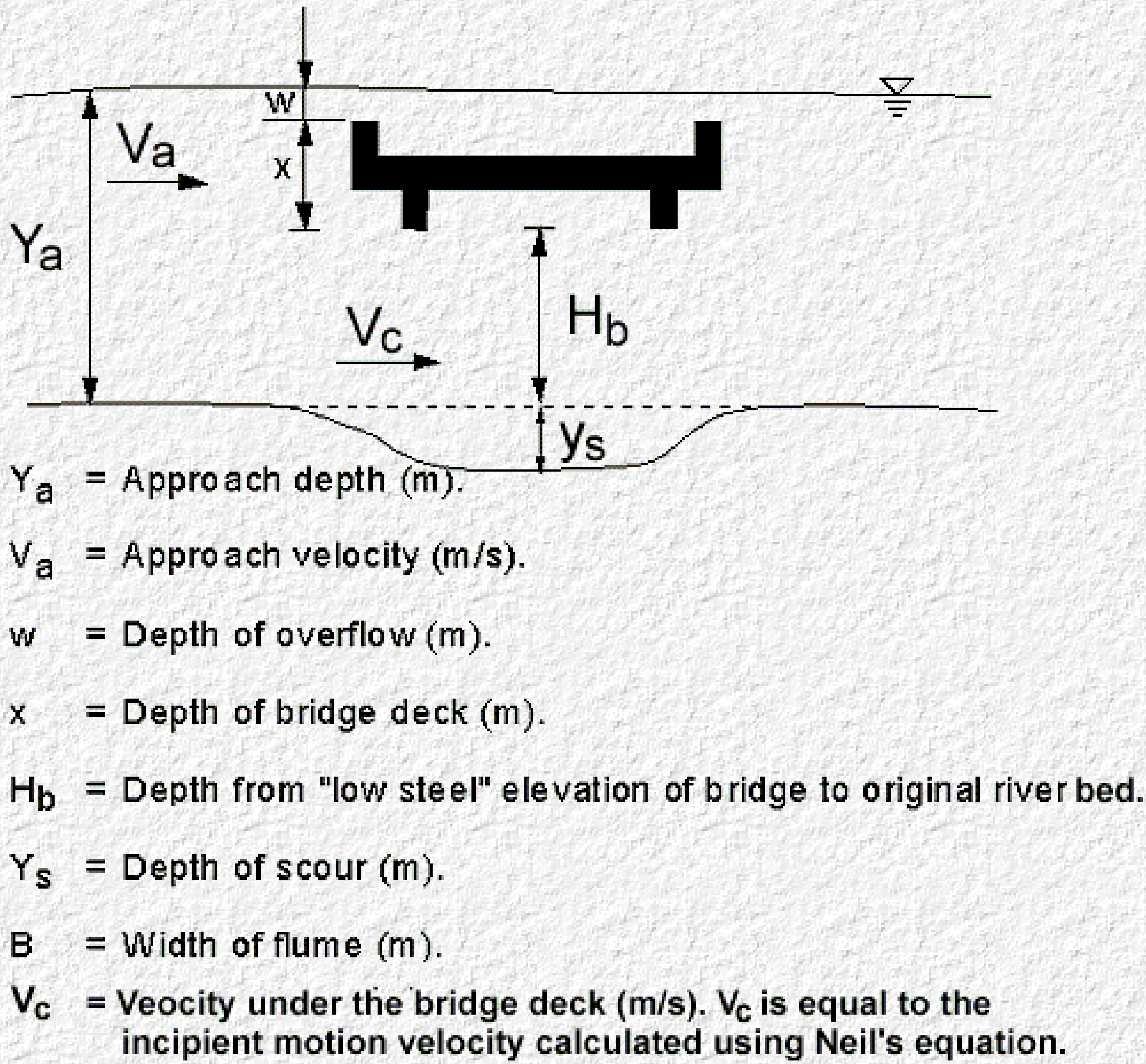

Figure 6. Definition Sketch for Pressure Flow Deck Scour

Chang (1995) analyzed the same data in search of a more general adjustment factor that could be extended into the live-bed zone. He reasoned that the V_a/V_c velocity ratio should not be a significant factor in the live-bed zone, and he determined that there should be two adjustment coefficients that could be applied to the assumed unit discharge that passes under the bridge deck to account for the distorted velocity distribution. For the clear-water zone his equation becomes:

$$H_b + Y_s = \frac{C_q\, q_{br}}{V_c}$$

where:

$$C_q = \frac{1}{C_f * C_c}$$

$c_f = 1.5\ N_f^{0.43} \leq 1.0$

and

$$C_c = \sqrt{0.10\left(\frac{H_b}{(y_a - w)} - 0.56\right) + 0.79} \leq 1.0$$

C_f and C_c are Froude number and vertical contraction corrections respectively and neither should exceed 1.0.

Figure 7 and Figure 8 illustrate the predicted versus measured deck scour component for the Umbrell method and Chang method, respectively, as described by Equation 1 and Equation 2 above. Since all of the available data was clear-water data, it was anticipated that the Umbrell method would better represent this data set, but the Chang correction factor has the potential of application under live-bed conditions.

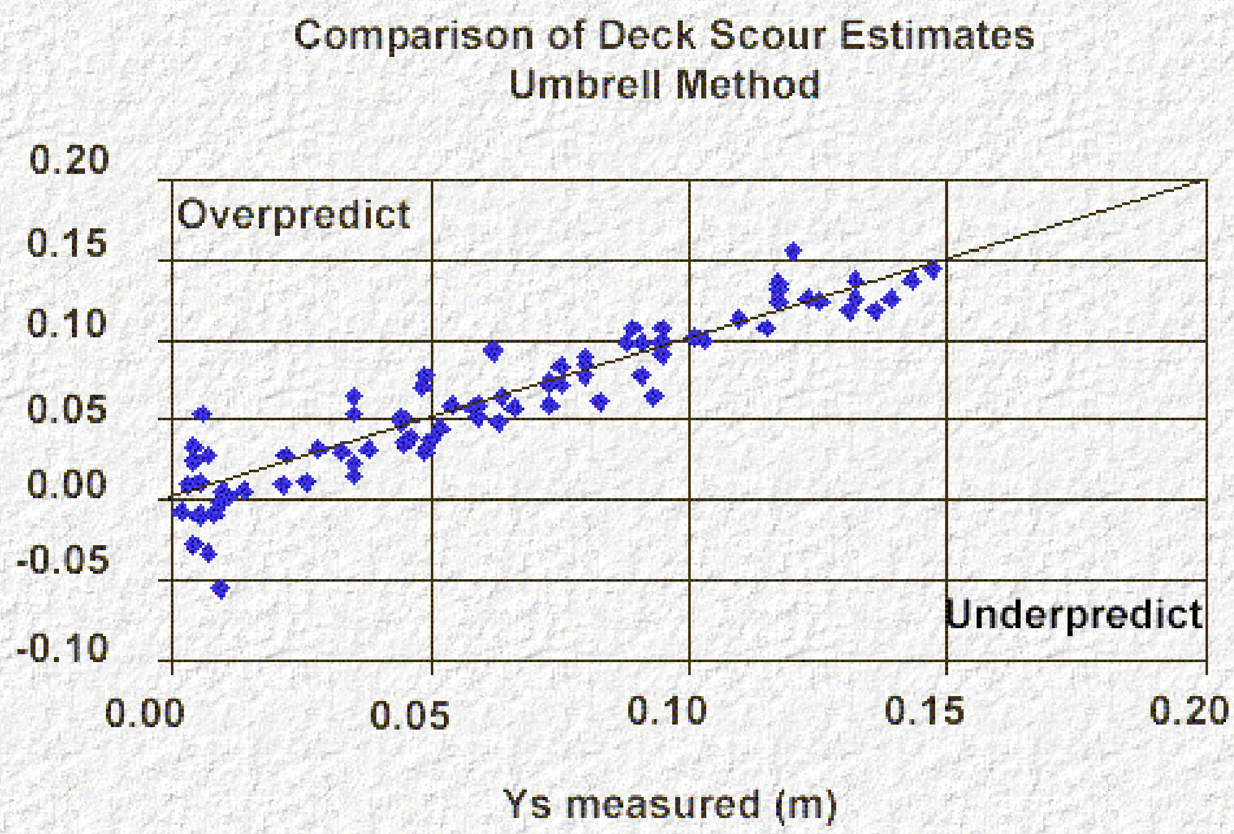

Figure 7. Predicted versus Measured Deck Scour Using Umbrell's Equation for Clear-water Conditions

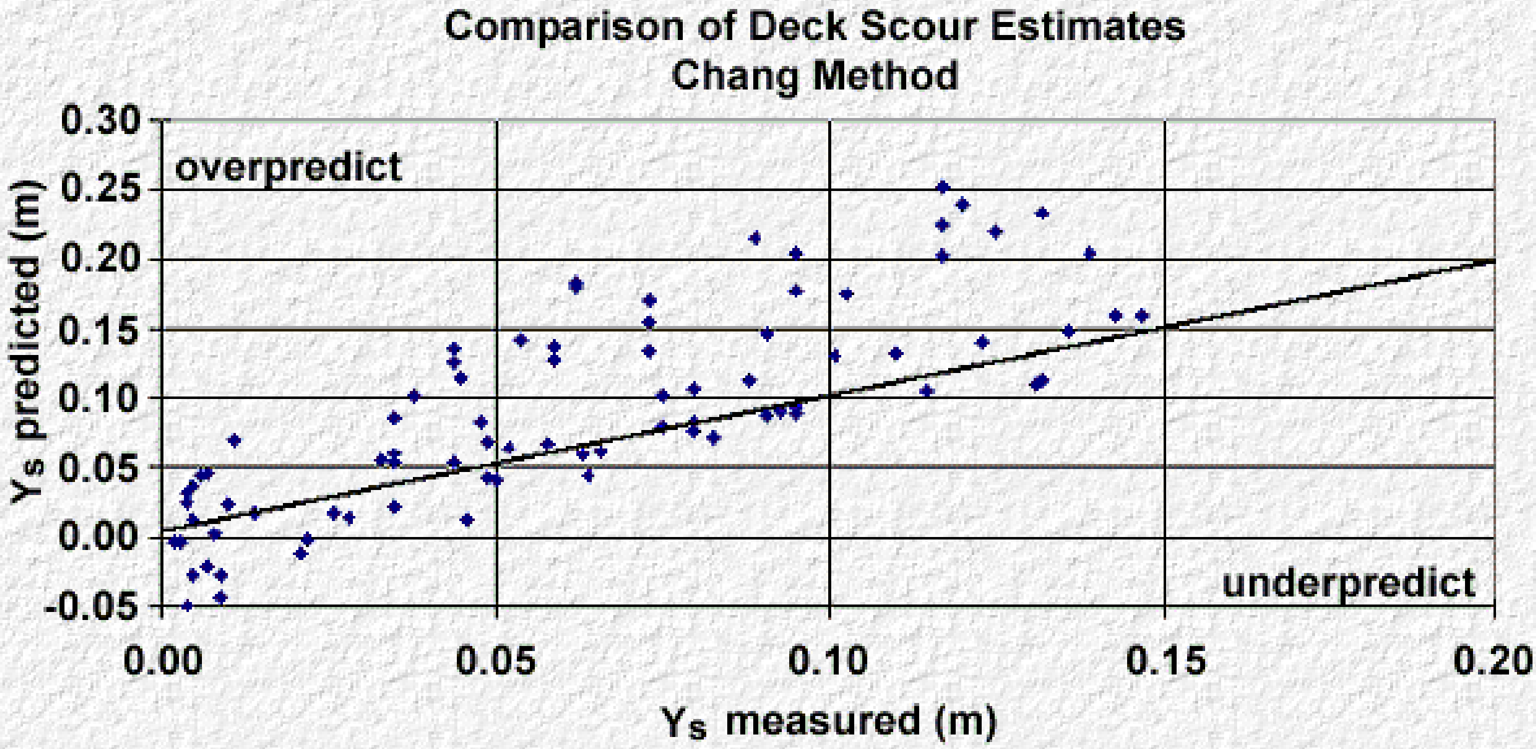

Figure 8. Predicted versus Measured Deck Scour Using Chang's Correction Coefficients

The big question is how to apply these results to live-bed conditions. One philosophy is to use either of the clear-water equations in the live-bed range and recognize that it will be an upper limit because the incoming sediment which replaces the outgoing sediment in equilibrium conditions would be ignored. If this results in an unreasonably large value of deck scour, then one should use Chang's C_q correction to adjust the unit discharge through the bridge opening and compute the size opening needed to balance the incoming bed load with the computed outgoing bed load for the adjusted discharge through the bridge opening only. The arithmetic is not as clean but the procedure is the same as Laursen used to develop the classic long contraction equations.

Data is urgently needed to verify or replace this suggested procedure for live-bed conditions. At the writing of this paper there is an ongoing comprehensive investigation at CSU by Arneson (7) to resolve questions of pressure-flow scour under live-bed conditions.

B-7 Conclusions

Although the cumulative effects of pressure flow can be incorporated into a gross correction to the pier scour estimate, the deck scour component and the pier scour component can and should be estimated separately and added.

The pier scour component can be estimated without significant adjustment if the appropriate velocity is used in the scour equations. That velocity need not exceed the incipient motion velocity, V_c, for clear-water scour.

Two equations are given for estimating the deck scour component under clear-water conditions where V_a is less than V_c. These equations can be used conservatively to compute deck scour under live-bed conditions but in some cases they may yield overly conservative results. An alternate approach using the discharge correction coefficients derived by Chang and a sediment discharge balance is suggested for live-bed conditions.

Additional research is urgently needed for pressure-flow scour experiments under live-bed conditions. Both of the studies described in this paper have been limited to clear-water experiments. An ongoing study at CSU is addressing the live-bed conditions.

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Appendix C : HEC 18

WSPRO Input and Output for Example Scour Problem

[Go to Appendix D](#)

C-1 Input Data for Chapter 4 Example Problem

APPENDIX C

WSPRO INPUT AND OUTPUT FOR EXAMPLE PROBLEM
INPUT DATA FOR CHAPTER 4 EXAMPLE PROBLEM

Input parameters

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*f
T1        WORKSHOP PROBLEM - SCOUR CREEK - METRIC CONVERSION
T2        ESTIMATING SCOUR AT BRIDGES - COMPUTER SIMULATION
T3        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
*
SI        1
*
Q         849.51
SK        0.002
*
XS   EXIT  228.6 * * * .002
GR        0,5.79  30.48,4.57  60.96,3.35  152.4,3.28  274.32,3.05  335.28,2.74
GR        370.33,1.68  381.00,1.49  396.24,0.93  411.48,1.48  422.15,1.55
GR        457.2,2.74  518.16,3.05  640.08,3.28  731.52,3.35  762.00,4.57
GR        792.48,5.79
N           0.042     0.032     0.042
SA               335.28    457.2
*
XS   FULLV 426.72
*
BR   BRDG  426.72
BL 1       198.12   335.28   457.2
BC         5.49
CD         3  15.24  2  6.71
AB         2
PD 0       1.72  9.14   6
N          0.042   0.032
SA            335.28
*
XS   APPR  640.08
*
HP 2 BRDG  4.23  1  4.23  849.51
HP 1 BRDG  4.15  1  4.15
HP 2 APPR  5.27  1  5.27  849.51
HP 1 APPR  5.27  1  5.27
*
EX
ER
```

C-2 Output Data for Chapter Example Problem

OUTPUT DATA FOR CHAPTER EXAMPLE PROBLEM

```
Line #   Input parameters
        ***********************  W S P R O  ***************************
          Federal Highway Administration  -  U. S. Geological Survey
                 Model for Water-Surface Profile Computations.
             Run Date & Time: 10/26/94  1:55 pm     Version V081594
              Input File: scourcrm.dat   Output File: scourcrm.lst
       *---------------------------------------------------------------*
    *F
     ***                   Input Data In Free Format                      ***

    T1         WORKSHOP PROBLEM - SCOUR CREEK - METRIC CONVERSION
    T2         ESTIMATING SCOUR AT BRIDGES - COMPUTER SIMULATION
    T3         CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
    SI         1
                         Metric (SI) Units Used in WSPRO
                    Quantity     SI Unit              Precision
                  ------------ -------------------- ----------
                   Length       meters                  0.001
                   Depth        meters                  0.001
                   Elevation    meters                  0.001
                   Widths       meters                  0.001
                   Velocity     meters/second           0.001
                   Discharge    cubic meters/second     0.001
                   Slope        meter/meter             0.001
                   Angles       degrees                 0.01
                  ------------ -------------------- ----------
    Q          849.51
     ***   Processing Flow Data; Placing Information into Sequence  1   ***
    SK         0.002
       ************************  W S P R O  ***************************

             *--------------------------------------------------*
             *      Starting To Process Header Record EXIT       *
             *--------------------------------------------------*
    XS   EXIT  228.6 * * * .002
    GR         0,5.79  30.48,4.57  60.96,3.35  152.4,3.28  274.32,3.05  335.28,2.74
    GR         370.33,1.68  381.00,1.49  396.24,0.93  411.48,1.48  422.15,1.55
    GR         457.2,2.74  518.16,3.05  640.08,3.28  731.52,3.35  762.00,4.57
    GR         792.48,5.79
    N            0.042     0.032     0.042
    SA                 335.28    457.2

     ***   Completed Reading Data Associated With Header Record EXIT     ***
     ***   Storing Header Data In Temporary File As Record Number  1     ***

     ***               Data Summary For Header Record EXIT               ***
     SRD Location:       229.   Cross-Section Skew:    .0   Error Code   0
     Valley Slope:    .00200    Averaging Conveyance By Geometric Mean.
     Energy Loss Coefficients ->   Expansion:    .50    Contraction:    .00

                           X,Y-coordinates (17 pairs)
         X          Y          X          Y          X          Y
    ---------- ---------- ---------- ---------- ---------- ----------
         .000      5.790     30.480      4.570     60.960      3.350
      152.400      3.280    274.320      3.050    335.280      2.740
      370.330      1.680    381.000      1.490    396.240       .930
      411.480      1.480    422.150      1.550    457.200      2.740
      518.160      3.050    640.080      3.280    731.520      3.350
      762.000      4.570    792.480      5.790
    ---------- ---------- ---------- ---------- ---------- ----------
                     Minimum and Maximum X,Y-coordinates
      Minimum X-Station:         .000  ( associated Y-Elevation:     5.790 )
      Maximum X-Station:      792.480  ( associated Y-Elevation:     5.790 )
      Minimum Y-Elevation:       .930  ( associated X-Station:     396.240 )
      Maximum Y-Elevation:      5.790  ( associated X-Station:     792.480 )

  Subarea Breakpoints (NSA =    3):
         335.     457.
```

```
Roughness Coefficients (NSA =  3):
        .042    .032    .042
          *-----------------------------------------------*
          *      Finished Processing Header Record EXIT      *
          *-----------------------------------------------*
     ************************ W S P R O **************************

          *-----------------------------------------------*
          *      Starting To Process Header Record FULLV      *
          *-----------------------------------------------*

  XS   FULLV 426.72

   ***   Completed Reading Data Associated With Header Record FULLV   ***
   ***   No Roughness Data Input, Propagating From Previous Section   ***
   ***   Storing Header Data In Temporary File As Record Number  2    ***

   ***             Data Summary For Header Record FULLV               ***

   SRD Location:      427.   Cross-Section Skew:     .0   Error Code   0
   Valley Slope:    .00200    Averaging Conveyance By Geometric Mean.
   Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                     X,Y-coordinates (17 pairs)
        X          Y            X          Y            X          Y
   ---------- ----------   ---------- ----------   ---------- ----------
        .000      6.186       30.480      4.966       60.960      3.746
     152.400      3.676      274.320      3.446      335.280      3.136
     370.330      2.076      381.000      1.886      396.240      1.326
     411.480      1.876      422.150      1.946      457.200      3.136
     518.160      3.446      640.080      3.676      731.520      3.746
     762.000      4.966      792.480      6.186
   ---------- ----------   ---------- ----------   ---------- ----------

                    Minimum and Maximum X,Y-coordinates
    Minimum X-Station:        .000  ( associated Y-Elevation:    6.186 )
    Maximum X-Station:     792.480  ( associated Y-Elevation:    6.186 )
    Minimum Y-Elevation:     1.326  ( associated X-Station:    396.240 )
    Maximum Y-Elevation:     6.186  ( associated X-Station:    792.480 )

Subarea Breakpoints (NSA =   3):
       335.    457.

Roughness Coefficients (NSA =  3):
        .042    .032    .042

          *-----------------------------------------------*
          *      Finished Processing Header Record FULLV      *
          *-----------------------------------------------*
     ************************ W S P R O **************************

          *-----------------------------------------------*
          *      Starting To Process Header Record BRDG      *
          *-----------------------------------------------*
  BR   BRDG  426.72
  BL   1       198.12   335.28   457.2
  BC           5.49
  CD           3  15.24  2  6.71
  AB           2
  PD  0        1.72  9.14   6
  N            0.042   0.032
  SA              335.28

   ***   Completed Reading Data Associated With Header Record BRDG    ***
   ***   Storing Header Data In Temporary File As Record Number  3    ***

   ***             Data Summary For Header Record BRDG                ***

   SRD Location:      427.   Cross-Section Skew:     .0   Error Code   0
```

```
     Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
     Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00
```



```
                     X,Y-coordinates (13 pairs)
        X          Y           X          Y           X          Y
    ---------- ----------  ---------- ----------  ---------- ----------
      263.788      5.490     267.852      3.458     274.319      3.446
      335.279      3.136     370.329      2.076     380.999      1.886
      396.239      1.326     411.479      1.875     422.149      1.946
      457.199      3.136     457.200      3.136     461.908      5.490
      263.788      5.490
    ---------- ----------  ---------- ----------  ---------- ----------
                 Minimum and Maximum X,Y-coordinates
     Minimum X-Station:     263.788  ( associated Y-Elevation:    5.490 )
     Maximum X-Station:     461.908  ( associated Y-Elevation:    5.490 )
     Minimum Y-Elevation:     1.326  ( associated X-Station:    396.239 )
     Maximum Y-Elevation:     5.490  ( associated X-Station:    263.788 )

 Subarea Breakpoints (NSA =   2):
        335.
 Roughness Coefficients (NSA =  2):
         .042    .032

 Discharge coefficient parameters:
  BRTYPE   BRWDTH   EMBSS    EMBELV   USERCD
    3       15.2     2.00      6.71   ******

 Pressure flow elevations:  AVBCEL =     5.49   PFELEV =     5.49

 Abutment parameters:
   ABSLPL   ABSLPR   XTOELT   YTOELT   XTOERT   YTOERT
     2.0   ******    267.9      3.5    457.2      3.1

 Bridge Length and Bottom Chord component input data:
   BRLEN   LOCOPT   XCONLT   XCONRT   BCELEV    BCSLP    BCXSTA
   198.1      1.     335.     457.     5.49   ******   *******

 Pier Data:  Number   1 Pier/Pile Code: 0.
       ELEV  WDTH  #P/P        ELEV  WDTH  #P/P        ELEV  WDTH  #P/P
       1.72   9.1   6.00

            *-------------------------------------------------*
            *      Finished Processing Header Record BRDG      *
            *-------------------------------------------------*
      **************************  W S P R D  ***************************

            *-------------------------------------------------*
            *      Starting To Process Header Record APPR      *
            *-------------------------------------------------*

   XS   APPR  640.08

    ***   Completed Reading Data Associated With Header Record APPR    ***
    ***   No Roughness Data Input, Propagating From Previous Section   ***
    ***   Storing Header Data In Temporary File As Record Number  4    ***

    ***              Data Summary For Header Record APPR               ***

    SRD Location:      640.   Cross-Section Skew:    .0   Error Code   0
    Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
    Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                     X,Y-coordinates (17 pairs)
        X          Y           X          Y           X          Y
    ---------- ----------  ---------- ----------  ---------- ----------
         .000      6.613      30.479      5.393      60.959      4.173
      152.399      4.103     274.319      3.873     335.279      3.563
```

```
         370.329        2.503        380.999        2.313        396.239        1.753
         411.479        2.302        422.149        2.373        457.199        3.563
         518.159        3.873        640.079        4.103        731.519        4.173
         761.999        5.393        792.479        6.613
       ----------   ----------   ----------   ----------   ----------   ----------
```



```
                      Minimum and Maximum X,Y-coordinates
     Minimum X-Station:         .000  ( associated Y-Elevation:     6.613 )
     Maximum X-Station:      792.479  ( associated Y-Elevation:     6.613 )
     Minimum Y-Elevation:      1.753  ( associated X-Station:     396.239 )
     Maximum Y-Elevation:      6.613  ( associated X-Station:     792.479 )

 Subarea Breakpoints (NSA =    3):
        335.     457.

 Roughness Coefficients (NSA =  3):
         .042     .032     .042

 Bridge datum projection(s):  XREFLT  XREFRT  FDSTLT  FDSTRT
                              ******* ******* ******* *******

            *-----------------------------------------------------*
            *      Finished Processing Header Record APPR         *
            *-----------------------------------------------------*
      *************************  W S P R O  ***************************

    HP  2 BRDG  4.23  1  4.23  849.51
    HP  1 BRDG  4.15  1  4.15
    HP  2 APPR  5.27  1  5.27  849.51
    HP  1 APPR  5.27  1  5.27
    EX

           *=====================================================*
           *      Summary of Boundary Condition Information      *
           *=====================================================*

               Reach       Water Surface   Friction
        #    Discharge       Elevation       Slope          Flow Regime
       --    ---------     -------------   --------     --------------------
        1       849.51        ********        .0020         Sub-Critical
       --    ---------     -------------   --------     --------------------

           *=====================================================*
           *        Beginning  1 Profile Calculation(s)          *
           *=====================================================*

      *************************  W S P R O  ***************************

                           WSEL      VHD         Q         AREA       SRDL       LEW
                           EGEL       HF         V           K        FLEN       REW
                           CRWS       HO       FR #         SF       ALPHA       ERR
                         --------- ------  ---------- ---------- ---------  ---------
 Section: EXIT             3.832   .173     849.509    622.871       .000     48.894
 Header Type: XS           4.006   .000       1.364   18992.99       .000    743.584
 SRD:     228.600          3.615   .000        .622      .0000      1.830       .000

 Section: FULLV            4.231   .172     849.509    624.430    198.119     48.837
 Header Type: FV           4.404   .395       1.360   19053.13    198.119    743.642
 SRD:     426.719          4.011   .000        .620      .0020      1.828       .002

         <<< The Preceding Data Reflect The "Unconstricted" Profile >>>

 Section: APPR             4.658   .172     849.509    624.574    213.360     48.829
 Header Type: AS           4.830   .424       1.360   19059.62    213.360    743.648
 SRD:     640.080          4.438   .000        .620      .0020      1.828       .002
```

```
          <<< The Preceding Data Reflect The "Unconstricted" Profile >>>

           <<< The Following Data Reflect The "Constricted" Profile >>>
                 <<< Beginning Bridge Hydraulics Computations >>>

 Section: BRDG          4.151   .769    849.509    294.018    198.119   266.464
 Header Type: BR        4.921   .620      2.889   12559.79    198.119   459.231
 SRD:     426.719       3.990   .293      1.004      .0020      1.806      .000

 Specific Bridge Information    C     P/A   PFELEV     BLEN      XLAB      XRAB
 Bridge Type 3   Flow Type 1 ------ ----- -------- -------- -------- --------
 Pier/Pile Code  0            .7441  .034    5.489   198.120  267.851  457.197
 -------------------------- ------ ----- -------- -------- -------- --------
```



```
 Section: APPR          5.268   .050    849.509   1058.158    198.120    33.581
 Header Type: AS        5.318   .323       .802   39088.53    213.359   758.896
 SRD:     640.080       4.438   .074       .263      .0020      1.534     -.003

                Approach Section APPR  Flow Contraction Information
                M( G )   M( K )     KQ       XLKQ     XRKQ     OTEL
               -------- -------- --------- -------- -------- --------
                  .722     .426   22535.5  271.518  463.594    5.175
               -------- -------- --------- -------- -------- --------

                  <<< End of Bridge Hydraulics Computations >>>

      ************************** W S P R O ****************************

    ***    Beginning Velocity Distribution For Header Record BRDG      ***
           SRD Location:     426.720        Header Record Number  3

        Water Surface Elevation:       4.230              Element # 1
        Flow:      849.510   Velocity:  2.75   Hydraulic Depth:    1.600
        Cross-Section Area:     309.17          Conveyance:     13531.24
           Bank Stations ->  Left:     266.307    Right:     459.388

 X STA.        266.3      305.8      332.9      348.6      358.2      366.0
   A( I )            32.8       27.5       19.8       15.8       14.9
   V( I )            1.29       1.54       2.15       2.68       2.86
   D( I )             .83       1.01       1.26       1.64       1.91

 X STA.        366.0      372.2      378.1      383.5      388.3      392.6
   A( I )            13.2       13.2       12.5       12.1       11.8
   V( I )            3.22       3.21       3.39       3.50       3.61
   D( I )            2.11       2.24       2.35       2.52       2.69

 X STA.        392.6      396.8      400.8      405.2      410.1      415.4
   A( I )            11.6       11.4       11.7       12.2       12.6
   V( I )            3.65       3.73       3.62       3.47       3.38
   D( I )            2.82       2.84       2.66       2.49       2.35

 X STA.        415.4      421.0      427.2      434.3      443.5      459.4
   A( I )            12.8       13.8       14.3       15.7       19.4
   V( I )            3.32       3.09       2.97       2.71       2.19
   D( I )            2.31       2.22       1.99       1.71       1.22


      ************************** W S P R O ****************************

    ***    Compute Cross-Section Properties For Header Record BRDG      ***
            SRD Location:     426.720        Header Record Number  3

 Water     S  Cross      Cross                  Bank Station
 Surface   A  Section    Section  Top   Wetted  ------------  Hydrlic  Critical
 Elevation #  Conveyance Area(s) Width   Pmtr    Left  Right   Depth     Flow
 --------- -- ---------- ------- ------ ------- ------ ------ -------- ---------
            1    1208.24    57.   68.8   68.98                    .834    164.12
```

```
                2    11333.03     236.  123.9  124.25                      1.906    1021.92
         4.150       12541.26     294.  192.8  193.22  266.5  459.2        1.523    1052.71
      --------- -- ---------- ------- ------ ------- ------ ------ ------- ---------
           ***************************  W S P R O  *****************************

        ***    Beginning Velocity Distribution For Header Record APPR       ***
               SRD Location:     640.080        Header Record Number  4

            Water Surface Elevation:        5.270              Element # 1
            Flow:      849.510    Velocity:    .80  Hydraulic Depth:    1.460
            Cross-Section Area:    1059.34          Conveyance:     39151.16
               Bank Stations ->  Left:      33.541    Right:     758.937

      X STA.        33.5       124.4       186.1       242.1       290.5       330.4
        A( I )              86.2        72.9        71.9        67.3        63.1
        V( I )               .49         .58         .59         .63         .67
        D( I )               .95        1.18        1.28        1.39        1.58

      X STA.       330.4       352.8       366.9       378.4       388.4       396.9
        A( I )              42.7        34.7        32.2        30.5        28.9
```



```
        V( I )               .99        1.22        1.32        1.39        1.47
        D( I )              1.91        2.45        2.80        3.05        3.38

      X STA.       396.9       405.5       415.4       426.5       440.0       462.5
        A( I )              28.5        30.2        32.0        33.9        43.5
        V( I )              1.49        1.41        1.33        1.25         .98
        D( I )              3.34        3.03        2.88        2.52        1.93

      X STA.       462.5       501.9       549.6       604.8       668.4       758.9
        A( I )              62.2        66.4        71.0        75.2        85.8
        V( I )               .68         .64         .60         .57         .49
        D( I )              1.58        1.39        1.29        1.18         .95


           ***************************  W S P R O  *****************************

        ***    Compute Cross-Section Properties For Header Record APPR      ***
               SRD Location:     640.080        Header Record Number  4

      Water      S  Cross       Cross                      Bank Station
      Surface    A  Section     Section  Top    Wetted  -------------  Hydrlic  Critical
      Elevation  #  Conveyance  Area(s)  Width   Pmtr    Left   Right   Depth     Flow
      --------- -- ---------- ------- ------ ------- ------ ------ ------- ---------
                 1   10075.62     370.  301.7  301.76                      1.225    1281.66
                 2   18999.92     320.  121.9  121.98                      2.622    1622.42
                 3   10075.62     370.  301.7  301.76                      1.225    1281.66
         5.270       39151.16    1059.  725.4  725.50    33.5  758.9       1.460    3237.29
      --------- -- ---------- ------- ------ ------- ------ ------ ------- ---------

        ER

        *******************  Normal end of WSPRO execution.  ******************
        ****************  Elasped Time:    0 Minutes   0 Seconds   ***************
```

Go to Appendix D

Appendix D : HEC 18

Hydraulics of Tidal Bridges, Maryland SHA's Procedure

Go to Appendix E

D-1 Introduction

The Maryland SHA conducts hydraulic studies for all proposed new structures over tidal waters. In addition, the SHA is presently in the process of rating its 251 on-system tidal bridges for vulnerability to scour damage. This appendix outlines the methods used to develop the hydraulic analysis of proposed and existing tidal bridges.

The following general principles have evolved as the SHA has gained experience in evaluating tidal bridges:

- The SHA concurs with the observations of C.R. Neill (Reference 4) that "rigorous analysis of tidal crossings is difficult **and is probably unwarranted in most cases** but in important cases consideration should be given to enlisting a specialist in tidal hydraulics".
- New structures over tidal waters are normally designed to span the tidal channel and adjacent wetlands. Such designs do not significantly constrict the tidal flow, and consequently minimize the extent of contraction scour. The primary concern about scour is normally the extent of local pier scour, and in some cases protection of abutments and approach roads from local scour and/or wave ride-up.
- Currents of storm tides in unconstricted channels are usually on the order of 1 to 3 feet per second.
- The HEC 18 equation for pier scour can be expected to over-estimate the extent of local pier scour at tidal bridges with wide piers and low velocities of flow.

Almost all tidal bridges in Maryland are located on the Chesapeake Bay or on estuaries or inlets tributary to the Bay. Previous studies commissioned by FEMA (Reference 12) have defined the elevation of the 100-year and 500-year storm tide elevations throughout the bay area. Studies by the SHA have identified a storm tide period of 24 hours, based on measured historic storm tides on the bay.

With this information, and the hydrologic study of flood runoff from upland drainage areas, the SHA conducts hydraulic studies of most tidal bridges following Neill's method as outlined in FHWA Hydraulic Engineering Circular 18 (Reference 13). Special cases where this method does not apply are addressed later in the appendix.

D-2 Evaluating Existing Tidal Bridges

In order to develop a cost-effective method of rating the 251 existing tidal bridges, the SHA is in the process of working up a screening process to identify low risk bridges. The basic tool used in this screening process is the Classification System. This system will also be useful in determining the extent of study required for future tidal bridge projects.

D-3 Classification of Tidal Bridges

Following the guidance presented by Neill (Reference 4), tidal bridge are categorized based on geometric configurations of bays and estuaries and the flow patterns at the bridges into three main categories:

I. bridges in enclosed bays or lagoons,

II. bridges in estuaries, and

III. bridges across islands or an island and the mainland.

SHA has also classified the tidal waterways to take into account whether:

- there is a single inlet or multiple inlets,
- there is a planned or existing channel constriction at the bridge crossing,
- river flow or tidal flow predominates for the anticipated worst-case condition for scour, and
- tidal flow or wind establishes the anticipated worst-case condition for scour for Category III bridge crossings.

D-4 Category 1. Bridges in Enclosed Bays Across Bay Inlets

In tidal waterways of this type, runoff from upland watersheds is quite limited, and the flow at the bridge is affected mainly by the tidal flow.

For an enclosed bay with only one inlet, the tidal flow must enter and exit through the inlet, and the hydraulic analysis is relatively straightforward. If there are multiple inlets to the bay, special studies must be made to determine the portion of the tidal prism that flows through each inlet for the design conditions.

If a highway crossing constricts a tidal waterway, there is a significant energy loss at the structure which must be taken into account in the hydraulic analysis. SHA has developed several methods for evaluating the effect of the constriction on the flow (Reference 14). These methods involve routing the storm tide in order to calculate the differential head across the structure and the resulting depth and velocity of flow through the structure.

D-5 Category II. Bridges in Estuaries

Flow in estuaries consist of a combination of riverine (flood runoff) flow and tidal flow. The ratio of these flows varies depending upon the size of the upland drainage area, the surface are of the tidal estuary, the magnitude and frequency of the storm tide and the magnitude, frequency, shape and lag time of the flood hydrograph.

Group A includes those bridges over channels where the flow is governed by riverine flow (90% or more of the total flow).

Group B includes bridges on estuaries where the flow is affected by both riverine and tidal flow.

Group C includes bridges over estuaries where 90% or more of the flow consists of tidal flow. The hydraulic analysis of bridges in this category is similar to those in Category I.

Bridges in Category II are also subdivided with regard to whether the waterway is constricted in the same manner as Category I bridges.

D-6 Category III. Bridges Connecting Two Islands or an Island and the Mainland

The hydraulic analysis of each bridge in this category is unique, and no general guidelines have been developed for such locations. The effect of wind may become a primary factor to be considered in such locations. The analysis of such tidal problems should be undertaken by Engineers knowledgeable about tidal hydraulics.

D-7 The SHA Screening Process

The SHA is using the following process to rate tidal bridges for Item 113, Scour Critical Bridges:

1. The location of each bridge is plotted on USGS topographic maps or NOAA navigation charts. Preliminary information is collected on the tidal waterway, upland drainage basin the highway crossing using a Tidal Bridge Data and Analysis Worksheet.
2. A preliminary estimate is made of the depths and velocities of storm tides, taking into account the expected contribution to the flow of flood runoff from the upland drainage basin. (This is based on a variation of Neill's method).
3. The bridge is categorized in accordance with the criteria.
4. An SHA "Phase 2" study is made of each bridge. The bridge plans and files are reviewed, along with the Phase 1 Channel Stability Study conducted by the U.S.Geological Survey. This step may or may not include another bridge site inspection by the hydraulic engineers/interdisciplinary team.
5. The structure is rated for Item 113 on the basis of the foregoing information. Structures on good foundations with no history of scour will be rated as low risk when the preliminary hydraulic analysis indicates that the velocities of flow and anticipated scour is low. In those locations where estimated velocities are high, additional studies are planned to determine the degree of risk of scour damage.

D-8 Hydraulic Analysis of Category I and II Tidal Bridges

Hydraulic analysis of tidal waterways can be highly complex due to its unsteady, nonlinear and three-dimensional nature. The complexity is further enhanced by the uncertainty surrounding the interaction of tidal flows and runoff events. Several numerical, analytical and physical modeling techniques are available in literature to partially address the hydraulic complexity of tidal waterways. However, it should be noted that is impractical and expensive to utilize some of the more accurate analytical tools for scour evaluation and rating of a multitude of bridges. The Maryland State Highway Administration has been using simplified analytical methods to evaluate Category I bridges (in enclosed bays or lagoons) and Category II bridges (in estuaries). The analyses of bridges in these categories depends on whether or not the highway crossings constrict the tidal waterways.

D-8.1 Unconstricted Channels

The tidal flow rate through a channel that is relatively unconstricted by a bridge opening depends on the rate at which the bay side of the bridge is "filled" or "emptied", since the head differential between the ocean and bay sides of the bridge is expected to be very small. The procedure outlined in Reference 13 and Reference 4 can be used to compute the maximum discharge through the bridge opening as follows:

$$Q_{max} = \frac{3.14 \ VOL}{T}$$

where:

Q_{max} = maximum discharge in a tidal cycle, m^3/s
VOL = volume of water in the tidal prism between high and low tide levels, m^3
T = tidal period, seconds

Using the tidal flow rate, the velocities for scour evaluation can be determined using a hydraulic model, or by simply dividing the flow rate with the area of the bridge opening at mean elevation of the tidal wave being analyzed.

D-8.2 Constricted Channels

Tidal flow through a contracted bridge waterway opening may be treated as flow through an orifice, in which a substantial energy loss is encountered. Generally, the flow through an orifice is expressed in terms of the area of the waterway opening and the difference in the water-surface elevations across the contracted section as:

$$Q_o = C_d \ A_c \sqrt{2g(H_s - H_t)}$$

where:

Q_o = flow through the bridge (m^3/s)
C_d = discharge coefficient
A_c = bridge waterway cross-sectional area, (m^2)
H_s = water-surface elevation upstream of the bridge (m)
H_t = tidal elevation downstream of the bridge (m)
g = 9.81 m/s^2

Using the principle of continuity of flow, the discharge through a contracted section of a tidal estuary can be analyzed as follows:

- The Engineer must make an estimate of the amount of riverine flow (flood runoff) to be expected during the portion of the tidal period being analyzed, and how this flow may vary with time.
- The amount of tidal flow is determined from the change in the volume of water in the tidal

basin over a specified period of time. This is calculated by multiplying the surface area of the upstream tidal basin (A_s) by the drop in elevation over the specified time ($Q_{tide} = A_s \, dH_s/dt$)

- The total flow approaching the bridge is equal to the sum of the tidal flow and the riverine flow, and the total flow passing through the bridge is calculated from equation II.1. Equation II.2 is derived by setting these flows equal to each other:

$$Q + A_s \frac{dH_s}{dt} = C_d \; A_c \sqrt{2g(H_s - H_t)}$$

where:

Q = riverine flow (m^3/s), and
A_s = surface area of tidal basin upstream of the bridge (m^2)

Equation II.2 is solved by routing the combined tidal flow and riverine flow through the bridge. This involves a trial and error process as explained below. Equation II.2 may be rearranged into the form of equation II.3 for the time interval, $\Delta t = t_2 - t_1$, subscripts 2 and 1 representing the end and beginning of the time interval, respectively.

$$\frac{Q_1 + Q_2}{2} + \frac{A_{s1} + A_{s2}}{2} \, \frac{H_{s1} - H_{s2}}{\Delta t} = C_d \left(\frac{A_{c1} + A_{c2}}{2} \right) \sqrt{2g \left(\frac{H_{s1} + H_{s2}}{2} - \frac{H_{t1} + H_{t2}}{2} \right)}$$

For a given initial condition, t_1, all terms with subscript 1 are known. For $t=t_2$, the downstream tidal elevation (H_{t2}), riverine discharge (Q_2), and waterway cross-sectional area (A_{c2}) are also known or can be calculated from the tidal elevation. Only the water-surface elevation (H_{s2}) and the surface area (A_{s2}) of the upstream tidal basin remain to be determined. Since the surface area of the tidal basin is a function of the water-surface elevation, the elevation of the tidal basin at time t_2 (H_{s2}) is the only unknown term in equation II.3. Its value can be determined by trial-and-error to balance the values on the right and left sides of equation II.3.

The following steps are normally followed in carrying out this computation.

1. Determine the period and amplitude of the design tide(s). This establishes the time rate of change of the water-surface on the downstream side of the bridge.
2. Determine how the surface area of the tidal basin upstream of the bridge varies with elevation. This is accomplished by planimetering successive contour intervals and plotting the variation of surface area with elevation.
3. Determine how the waterway area of the bridge varies with elevation. This step is facilitated by use of a plot of bridge waterway area vs. elevation.
4. Make a judgment as to how to account for any upland runoff that is expected to occur during passage of the storm tide through the bridge.
5. Route the flows through the contracted waterway using equation II.3, and determine the maximum velocity of flow. This step is facilitated by use of a spreadsheet as illustrated in the following example problem.

There are various methods available for accomplishing the routing of the tide. In a spreadsheet or computer program, the variation with elevation of the tidal basin surface area and the bridge waterway area will need to be approximated by a mathematical equation. The following example illustrates the use of a spreadsheet routing program requiring a trial and error solution for each time interval. A more sophisticated procedure that reduces the steps required for the trial-and-error solution by utilizing a computer program is presented in Appendix A.

D-8.3 Example

Use of the procedure discussed above is demonstrated in this example problem for the tidal bridge carrying Maryland Route 286 over Back Creek in Cecil County. This bridge causes a severe contraction of the tidal flow and critical flow occurs under certain flow conditions. Because of this condition, the spreadsheet needed to be designed to calculate both orifice flow and critical flow and to provide guidance as to which flow condition should be used. In most cases, tidal bridges should not create a constriction narrow enough to cause the flow to pass through critical depth, and the spreadsheet computations should be less complex.

The process for determining the data needed for the flood routing procedure is presented below.

1. The worst condition for scour is found to occur during the ebbing of the storm tide when the tidal flow is augmented by flood runoff. The change of the water-surface elevation with time for the downstream side of the bridge due to the ebbing of the storm tide is determined from equation II.4 (See Equation 75 of Section 4.6.4 in Reference 13).

$$y = 1.75 \, \text{Cos}\left(2\pi \frac{t}{T}\right) + 1.75$$

where:

T = tidal period, selected as 24 h
A = 1.75 m, one-half of the tidal wave amplitude
y = tidal elevation (m)
t = time (h)

2. The relationship between the surface area of the tidal basin (upstream from the bridge) and the water-surface elevation is approximated by the following equation.

$$A_S = 0.635 \; {H_S}^{0.45} * 10^6$$

3. The waterway area of the bridge varies with the elevation. The waterway area becomes a constant value of 11.97 m^2 for elevations above 1.66 m. For elevations equal to or lower than 1.66 m, the waterway area, A_c, can be approximated by equation II.6.

$$A_c = 6.09 * (\text{Tidal Basin Elevation}) + 1.865$$

This equation is applicable above elevation 0.61 m.

4. The method of accounting for upland runoff must be determined on a case-by-case basis. Because of the uncertainty in estimating when the flood peak will occur during the passage of the ebb tide, the flood hydrograph was modified to represent a constant discharge equal to the average flood flow discharge. Since the duration of the flood hydrograph is about equal to the time required for the storm tide to ebb (12 h) it is assumed that the runoff starts at the same time that the tide begins to ebb.

5. The tidal routing method is illustrated in Table 1. This table was developed by using spreadsheet software. The routing procedure requires a trial and error approach to balance both sides of equation II.3. In the case of Back Creek, both the roadway and bridge are at an elevation of about 2.44 m and the 100-year storm tide will overtop the roadway. This condition was taken into account by the assumption that for the overtopping condition there will be a small head differential and therefore an insignificant amount of flow through the bridge. Because of this condition, the routing process is initiated at time t = 270 min when the tide elevation drops below the elevation of the roadway and bridge.

The discharge coefficient, C_d, is the product of the coefficient of contraction, C_c, and the velocity coefficient, C_v: $C_d = C_c * C_v$. The velocity coefficient is assumed to be 1.0 for this analysis. The area of flow in the downstream contracted section of the bridge is then equal to the area of the flow as it enters the bridge times the coefficient of contraction, C_c.

$$Q_o = C_d \ A_{upstream}\sqrt{2g\Delta H} = A_{downstream}\sqrt{2g\Delta H}$$

The downstream area of flow corresponding to the tidal elevation is used in the routing procedure for the orifice flow condition.

If the difference in hydraulic grade line across the contracted section exceeds one-third of the flow depth, upstream of the bridge (d), the flow will pass through critical depth. The discharge then will be limited to that corresponding to the critical flow condition, which can be expressed as:

$$Q_{cr} = A_{cr}\sqrt{g d_{cr}} = A_{cr}\sqrt{\frac{2}{3} g d}$$

where:

Q_{cr} = critical discharge (m^3/s)
A_{cr} = critical flow area (m^2)
d_{cr} = critical depth (m)
d = flow depth upstream of bridge
g = 9.81 m/s^2

The orifice equation no longer applies in this case; instead, the water-surface elevation is controlled by the critical discharge. The drop in the water surface of the tidal basin at the end of time Δt will be:

$$\Delta H_s = \Delta d = \frac{(Q_c - Q)\Delta t}{A_s}$$

where:

$\Delta H_s = \Delta d$ = drop in water-surface elevation (m)
Q_c = critical flow discharge (m^3/s)
Q = riverine flow (m^3/s)
A_s = surface area of the tidal basin (m^2)

If (Q_c - Q) is negative, it means that more water is flowing into the tidal basin than is flowing out through the bridge, and the water-surface elevation will rise in the tidal basin.

D-8.4 Explanation of Spreadsheet Columns

The spreadsheet columns in Table 1 are defined below. This method requires the Engineer to manually balance the tidal routing equation (equation II.3) for each time interval.

The bridge opening for the Route 286 structure is small; consequently, it creates a large head differential across the structure, and critical flow conditions develop during passage of the tidal flow. This condition of shifting controls requires several additional checks to be made in the spread sheet which may not be required for more typical tidal constrictions involving only minor contraction of the tide channel.

The spread sheet calculates the variables involved in balancing equation II.3. To do this, the spread sheet calculates the discharge due to orifice flow (Column R) and discharge due to critical flow conditions (Column T) and selects the smaller value of the two numbers for the value of Column V. By selecting values of the tidal basin elevation, Column F, The Engineer is able to find the correct value to balance equation II.3. This balance occurs when the same answer is obtained for Columns U and V.

In order to present the most significant aspects of the routing procedure in Table 1, certain less significant columns were omitted by using the "hiding" capability the spreadsheet. However, all of the Columns used in the analysis are described below:

Column	Column Description
A	Time, in minutes, from the beginning of the ebb tide. The start of the routing process is set at t = 270 min, when the water-surface elevation of the tide recedes below the elevation of the roadway.
B*	Routing time interval, selected as 15 minutes.
C	Downstream tide elevation, calculated by the using equation II.4. Cell C8 is designated as: 1.75*@COS(3.14*A8/720) + 1.75.
D	Waterway cross-sectional area as a function of the tide elevation: (a) A_c = 11.97 m^2. for Ht>1.66 m. (b) Use equation II.6 for 0.61<Ht<1.66 m. Cell D9 is designated: @IF(F9>= 1.66,11.97,(6.09*F9+ 1.865).
E*	Riverine flow, Q = 18 m^3/s, was used in this example for the reasons described in the text.
F	Water-surface elevation of the tidal basin upstream of the bridge. The first value in this column must be specified. All other values are estimated as a trial value to satisfy equation II.3. Note that column L is equal to the average value of Column F for the routing period.
G	Surface area of tidal basin. It is a function of the water-surface elevation upstream of the bridge as calculated from equation II.5. Cell G8 is designated: 0.635*(F8)^0.45.
H	Average (downstream) tide elevation. Cell H9 is designated: 6.09*H9+1.865.
I	Average waterway cross-sectional area. Cell I9 is designated: (D9+D8)/2.

J	Critical flow area under structure. This is written as an "IF" statement to account for full flow. Cell J8 is designated @IF((0.667*(L9+0.61)-0.61)<1.66,6.09*(0.667*(L9+0.61) -0.61)+1.865),11.97.
K	Average riverine flow. Cell J9 is designated: (E9+E8)/2.
L	Average water-surface elevation of the tidal basin. Cell K9 is designated: (F9+F8)/2.
M	Average surface area of the tidal basin. Cell M9 is designated: (G9+G8)/2.
N	Difference in water-surface elevations of the basin between time intervals. Cell N9 is designated: (+F8-F9)
O	Difference in water-surface elevation across the bridge. Cell N9 is designated: (+L9-H9).
P*	This column is not used.
Q	Velocity of flow based on the orifice equation (equation II.1) Cell Q9 is designated: @SQRT(19.62*(L9-H9))
R	Discharge based on the orifice equation, $Q = A_{downstream}$*V. Cell R9 is designated as: I9*Q9. Note that if the cell is shaded, it is not the controlling discharge used in Column V.
S	Critical velocity at the structure. Cell S9 is designated: @SQRT(9.81*0.667(L9+ 0.61)).
T	Discharge based on conditions of critical flow. Cell T9 is designated as: S9*J9.
U	Summation of the values on the left side of equation II.3. Cell U9 is designated: +K9 + 10^6*M9*N9/(B9*60).
V	Value of the right side of equation II.3. When the water-surface elevation (Column F) is assumed correctly, Cell V9 will equal Cell U9. An "IF" statement is used to select the correct discharges of Columns R and T, **based on critical depth**. Cell V9 is designated: @IF((H9+0.61)< =0.667*(L9+0.61),T9, R9).

NOTES:

1. The equations in the spread sheet were developed for the specific site conditions of the Back Creek structure. For other locations, different equations may be necessary to describe sub-critical and critical flow conditions

2. Columns marked with an asterisk are "hidden" columns in Table 1.

3. Portions of Columns R and T are shaded as an aid in identifying the flow condition at the structure. The spread sheet compares the two flow conditions - Column R, orifice flow and Column T, critical flow - and selects the lowest value for use in Column V. The unshaded or clear portions of these columns are the values that are used in Column V.

4. Great precision in balancing Columns U and V is probably not warranted. The Engineer may wish to determine an acceptable tolerance level for making this balance in order to limit the number of trials required. A tolerance of 5% or less is recommended.

5. The velocities computed in Columns Q and S are used for purposes of determining the rate of flow through the structure. The highest velocity in the structure may occur downstream from the control section, and can be computed separately for the worst flow condition.

6. The Table 1 spreadsheet serves the purpose of routing the combined tidal flow and flood runoff through the bridge. However, a hydraulic analysis should be performed using WSPRO, HEC-2, HY-8 or similar program to determine the actual water surface profile through the structure for a selected routing period.

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Tidal Flow Through a Contracted Bridge Opening,

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D-9 Appendix A-1

By rearranging equation II.3 using the approximation given in equation II.7, the following equation results:

$$H_{s2}^2 - (2k + 2H_{s1} + gp^2)H_{s2} + \left[(H_{s1} + K)^2 + gp^2(H_{t1} + H_{t2} - H_{s1})\right] = 0$$

where:

$$K = \frac{Q_1 + Q_2}{A_{s1} + A_{s2}} \Delta t2$$

and

$$p = \frac{A_{cd1} + A_{cd2}}{A_{s1} + A_{s2}} \Delta t$$

In equation A.3, the bridge flow cross-sectional areas are denoted using A_{cd} to clarify that they are computed based on downstream (tidal) water surface elevations. Equation A.1 is a quadratic equation with a solution for H_{s2} as follows:

$$H_{s2} = \frac{(2K + 2H_{s1} + gp^2) - \sqrt{(2K + 2H_{s1} + gp^2)^2 - 4\ [(H_{s1} + K)^2 + gp^2 (H_{t1} + H_{t2} - H_{s1})]}}{2}$$

In case of critical flow through the bridge ($\Delta H > d_{cr}$), the right hand side of equation II.3 is modified using equation II.8, resulting in the following:

$$\frac{Q_1 + Q_2}{2} + \frac{A_{s1} + A_{s2}}{2} \frac{H_{s1} - H_{s2}}{\Delta t} = \left(\frac{A_{cr1} + A_{cr2}}{2}\right)\sqrt{\frac{2}{3} g\left(\frac{H_{s1} + H_{s2}}{2} - Z_{br}\right)}$$

where Z_{br} = Elevation (m) of the bridge invert, and A_{cr} is based on upstream water surface elevation.

By rearranging equation A.5, the following equation results:

$$H_{s2}^2 - \left(2K + 2H_{s1} + \frac{gp_2^2}{3}\right)H_{s2} + \left[(H_{s1} + K)^2 - \frac{gp_2^2}{3}(H_{s1} - 2Z_{br})\right] = 0$$

where

$$p_2 = \frac{A_{cr1} + A_{cr2}}{A_{s1} + A_{s2}} \Delta t.$$

The quadratic form of equation A.1 and Equation A.6 allow quick convergence of solution for the water surface elevation upstream of the bridge (H_{s2}) at a given time, if the same is known for the previous time step. Therefore, by specifying the initial water surface elevation at a given time, and by utilizing other data (tidal amplitude, mean tidal elevation, tidal period, the variation of water surface area on the upstream side of the bridge as well as the cross-sectional area of the bridge in relation to water surface elevation) the water surface elevation at specified time increments can be computed. It is recommended that the specified time increments be small enough to avoid rapid variations that can cause computational difficulties. The computational steps can be accomplished using a computer program or a commercial spreadsheet software. The BASIC computer program (TIDEROUT.BAS) that follows the text can accomplish the computations shown in Table 1 much quicker, and without any need for fitting equations to describe waterway area and surface area as functions of water surface elevations. Instead, a table of planimetered values would be sufficient. The program us designed to be interactive, and can be executed, compiled or uncompiled, using a modern BASIC language software.

The same sample shown in Table 1 is executed using the computer program. The printout that follows shows the input and the output. The program only prints out the results at control condition (difference in water surface elevations vs. critical depth through bridge). The engineer should evaluate these results and determine if further computations are needed to establish the hydraulic parameters. For example, the exit velocity may be higher than the control velocity, and can be computed separately by the engineer if deemed necessary. The results compare favorably with the spreadsheet shown in Table 1. The slight differences in results are due to the refinement achieved through the quadratic equation formulation.

D-10 Program Listing

```
DECLARE FUNCTION CODES$ (CODE%)
DECLARE FUNCTION INTERP! (X!(), Y!(), N!, XX!)
CLS
PRINT " PROGRAM TIDEROUT.BAS"
PRINT " DEVELOPED BY: RAJA VEERAMACHANENI,"
PRINT " MARYLAND STATE HIGHWAY ADMINISTRATION"
PRINT " LANGUAGE: MICROSOFT QUICK BASIC (SUPPLIED WITH MS DOS 5.0)"
PRINT " PURPOSE:"
PRINT " To automate routing of tidal flow, in combination with"
PRINT " uniform fresh water flow through bridges over tide"
PRINT " influenced streams for scour analyses. This program"
PRINT " is intended to handle simple cases, with no weir flow"
PRINT " and other hydraulic complexities. Neither the developer nor"
PRINT " the Maryland State Highway Administration assume"
PRINT " responsibility for the results. The user must be an experienced"
PRINT " Hydraulic Engineer and should rely on his/her own judgement"
PRINT " to determine if this program's results are appropriate for"
PRINT " any particular use."
PRINT
PRINT "Hit any key to continue..."
DO WHILE INKEY$ = ""
LOOP
DIM XSU(50), YSU(50), XBR(50), YBR(50)
REM Change the following to 32.2 for US Customary Units.
GRAV = 9.81
CLSPRINT "IF YOU WANT TO PRINT THE INPUT & OUTPUT, TURN PRINTER ON"
PRINT "AND PRESS <CTRL> AND <PRINT SCREEN> BUTTONS SIMULTAEOUSLY"
PRINT
PRINT "PRESS ANY KEY TO CONTINUE"
DO WHILE INKEY$ = ""LOOP
CLS
INPUT "Enter Starting Time in hours:"; A1
INPUT "Enter Starting Head water elevation in meters"; D1
INPUT "Enter time step in hours:"; B1
```

```
INPUT "Enter Ending time in hours:"; TMAX
INPUT "Enter Tidal Amplitude in meters:"; AMP
INPUT "Enter Mean tidal elevation in meters:"; Z
INPUT "Enter Tidal Period in hours:"; TP
INPUT "Enter River Discharge in cubic meters per second:"; QRIVER
REM Setting maximum iterations to 3
IMAX = 3
REM
PRINT
PRINT "READ RATING TABLE FOR BASIN SURFACE AREA"
INPUT "ENTER NUMBER OF DATA PAIRS"; NSU
PRINT
PRINT "Enter each Data Pair";
PRINT " in ascending order of elevations separated by comma:";
PRINT " Elevation(m), Basin Surface Area(sq. m) [eg. -0.61,0]"PRINT
FOR II = 1 TO NSU
PRINT "DATA PAIR NUMBER "; II; "= ";
INPUT XSU(II), YSU(II)NEXT II
REM
PRINT
PRINT "READ RATING TABLE FOR BRIDGE OPENING AREA"
PRINT
INPUT "ENTER NUMBER OF DATA PAIRS"; NBR
PRINT
PRINT "Enter each Data Pair in ascending order";
PRINT " of elevations separated by comma:";
PRINT " Elevation, Bridge Cross-sectional Area (eg. -0.61,0)"
PRINT
PRINT "!!! IMPORTANT !!! : "
PRINT "THE FIRST PAIR MUST BE FOR THE BRIDGE INVERT!!!"
PRINT
FOR II = 1 TO NBR
PRINT "DATA PAIR NUMBER "; II; "= ";INPUT XBR(II), YBR(II)
NEXT IIREM
REM ASSIGN BRIDGE INVERT AS THE 1ST POINT OF XBR()
ZBR = XBR(1)BAREA = YBR(NBR)
HTFULL = XBR(NBR)
REM Preparing for first iteration

A2 = A1 + B1
B2 = B1
E1 = QRIVER
F1 = INTERP(XSU(), YSU(), NSU, D1)
C1 = Z + AMP * COS(2 * 3.14159 * A1 / TP)
C2 = Z + AMP * COS(2 * 3.14159 * A2 / TP)
G1 = INTERP(XBR(), YBR(), NBR, C1)
G2 = G1
E2 = QRIVER
F2 = F1
H1 = (E1 + E2) * B1 * 3600 / (F1 + F2)
I1 = (G1 + G2) * B1 * 3600 / (F1 + F2)
J1 = 1
K1 = 2 * (H1 + D1) + GRAV * I1 * I1
L1 = (D1 + H1) ^ 2 + GRAV * I1 * I1 * (C1 + C2 - D1)
M1 = D1 - C1
REM Checking for flood tidal flow
IF M1 < 0 THEN PRINT "FLOW DIRECTION REVERSED --- STOPPING"
STOP
END IF
N1 = SQR(2 * GRAV * M1)
O1 = G1 * N1
P1 = .5 * (K1 - SQR(K1 * K1 - 4 * J1 * L1)) / J1
PRINT
```

```
PRINT "TIME TIDE El. BASIN El.
DISCHARGE VELOCITY FLOW CONTROL"
PRINT " (hrs) (m) (m) (cms) (m/s) "
REM FORMAT$ = "+###.## +###.## +###.## +######.## +####.###"
FORMAT$ = "+###.## +###.## +###.## +######.## +####.### &"
REM
REM Beginning iteration
CODE% = 0
CODEE$ = CODES$(CODE%)
PRINT USING FORMAT$; A1; C1; D1; O1; N1; CODEE$DO WHILE (A1 < TMAX)
FOR I = 1 TO IMAX
C2 = Z + AMP * COS(2 * 3.14159 * A2 / TP)
F2 = INTERP(XSU(), YSU(), NSU, D2)
FAVG = (F1 + F2) / 2
DAVG = (D1 + D2) / 2
E2 = QRIVER
G2 = INTERP(XBR(), YBR(), NBR, C2)
E2 = E1
H2 = (E1 + E2) * B2 * 3600 / (F1 + F2)
I2 = (G1 + G2) * B2 * 3600 / (F1 + F2)
J2 = J1
K2 = 2 * (H2 + D1) + GRAV * I2 * I2
L2 = (D1 + H2) ^ 2 + GRAV * I2 * I2 * (C1 + C2 - D1)
M2 = D2 - C2
IF M2 < 0 THEN
PRINT "FLOW DIRECTION REVERSED AT
T="; A2; " --- STOPPING"
STOP
END IF
MAVG = (M1 + M2) / 2
N2 = SQR(2 * GRAV * MAVG)
GAVG = (G1 + G2) / 2
O2 = GAVG * N2
P2 = .5 * (K2 - SQR(K2 * K2 - 4 * J2 * L2)) / J2
DEPBRI = DAVG - ZBR
CODE% = 1
IF ((DEPBRI / 3) - MAVG) < 0 THEN
REM
REM CHECK FOR CRITICAL FLOW THROUGH BRIDGE
CODE% = 2
DEPCRI = (2 / 3) * DEPBRI
NCRIT = SQR(GRAV * DEPCRI)
GCRIT = INTERP(XBR(), YBR(), NBR, (DEPCRI + ZBR))
OCRIT = GCRIT * NCRIT
HCRIT = H2
ICRIT = 2 * GCRIT * B2 * 3600 / FAVG
KCRIT = 2 * (HCRIT + D1) + GRAV * (ICRIT ^ 2) / 3
LCRIT = (D1 + HCRIT) ^ 2 - (D1 - 2 * ZBR) *
GRAV * (ICRIT ^ 2) / 3
REM
CODE% = 3
N2 = NCRITO2 = OCRIT
P2 = .5 * (KCRIT - SQR(KCRIT ^ 2 - 4 * J2 * LCRIT)) / J2
REM
END IF
D2P = D2
D2 = P2
IF I = IMAX THEN
REM FORMAT$ = "+###.## +###.## +###.## +######.## +####.###"
FORMAT$ = "+###.## +###.## +###.## +######.## +####.### &"CODEE$ = CODES$(CODE%)
PRINT USING FORMAT$; A2; C2; D2; O2; N2;
CODEE$
END IF
```

```
CODE% = 0
NEXT I
A1 = A2A2 = A1 + B2
B1 = B2
`

C1 = C2
D1 = D2
E1 = E2
F1 = F2
G1 = G2
H1 = H2
I1 = I2
J1 = J2
K1 = K2
M1 = M2
O1 = O2P1 = P2
LOOP
PRINT
PRINT "Normal End of Program"
END

FUNCTION CODES$ (CODE%)
CASE 0
CODES$ = "INITIAL CONDITION"
CASE 1 TO 2
CODES$ = "HEAD DIFFERENCE"
CASE 3
CODES$ = "CRITICAL DEPTH"
END SELECT
END FUNCTION

FUNCTION INTERP (X(), Y(), N, XX)
I = 1
IF XX <= X(1) THEN INTERP = Y(1): EXIT
FUNCTIONIF XX >= X(N) THEN INTERP = Y(N): EXIT
FUNCTION
DO WHILE ((I < N) OR (XX >= X(I)))
IF XX < X(I + 1) THEN EXIT DO
I = I + 1
LOOP
INTER = Y(I) + (XX - X(I)) * (Y(I + 1) - Y(I)) / (X(I + 1) - X(I))
INTERP = INTER
END FUNCTION
```

D-10.1 Printout From Tinderout.Bas

PROGRAM TIDEROUT.BAS
DEVELOPED BY: RAJA VEERAMACHANENI,
MARYLAND STATE HIGHWAY ADMINISTRATION
LANGUAGE: MICROSOFT QUICK BASIC (SUPPLIED WITH MS DOS 5.0)
PURPOSE:

To automate routing of tidal flow, in combination with uniform fresh water flow through bridges over tide influenced streams for scour analyses. This program is intended to handle simple cases, with no weir flow and other hydraulic complexities. Neither the developer nor the Maryland State Highway Administration assume responsibility for the results. The user must be an experienced Hydraulic Engineer and should rely on his/her own judgement to determine if this program's results are appropriate for any particular use.

Hit any key to continue...

IF YOU WANT TO PRINT #1, THE INPUT & OUTPUT, TURN PRINT #1,ER ON AND PRESS <CTRL> AND <PRINT #1, SCREEN> BUTTONS SIMULTAEOUSLY

PRESS ANY KEY TO CONTINUE
Enter Starting Time in hours:? 4.5
Enter Starting Head water elevation in meters? 2.438
Enter time step in hours:? .5
Enter Ending time in hours:? 14
Enter Tidal Amplitude in meters:? 1.75
Enter Mean tidal elevation in meters:? 1.75
Enter Tidal Period in hours:? 24
Enter River Discharge in cubic meters per second:? 18

READ RATING TABLE FOR BASIN SURFACE AREA
ENTER NUMBER OF DATA PAIRS? 5

Enter each Data Pair in ascending order of elevations separated by comma: Elevation(m), Basin Surface Area(sq. m) [eg. -0.61,0]

DATA PAIR NUMBER 1 =? -.61,0
DATA PAIR NUMBER 2 =? 0,300000
DATA PAIR NUMBER 3 =? 1,635000
DATA PAIR NUMBER 4 =? 2,867000
DATA PAIR NUMBER 5 =? 3,1041000

READ RATING TABLE FOR BRIDGE OPENING AREA

ENTER NUMBER OF DATA PAIRS? 5

Enter each Data Pair in ascending order of elevations separated by comma: Elevation, Bridge Cross-sectional Area (eg. -0.61,0)

!!! IMPORTANT !!! :
THE FIRST PAIR MUST BE FOR THE BRIDGE INVERT!!!

DATA PAIR NUMBER 1 =? -.61,0
DATA PAIR NUMBER 2 =? 0,2.79
DATA PAIR NUMBER 3 =? .61,5.58
DATA PAIR NUMBER 4 =? 1.22,9.29
DATA PAIR NUMBER 5 =? 1.66,11.99

TIME (hrs)	TIDE El. (m)	BASIN El. (m)	DISCHARGE (cm3)	VELOCITY (m/s)	FLOW CONTROL
+4.50	+2.42	+2.44	+7.18	+0.599	INITIAL CONDITION
+5.00	+2.20	+2.44	+18.84	+1.572	HEAD DIFFERENCE
+5.50	+1.98	+2.41	+30.67	+2.558	HEAD DIFFERENCE
+6.00	+1.75	+2.37	+38.60	+3.219	HEAD DIFFERENCE
+6.50	+1.52	+2.32	+43.23	+3.738	HEAD DIFFERENCE
+7.00	+1.30	+2.27	+43.65	+4.177	HEAD DIFFERENCE
+7.50	+1.08	+2.14	+41.19	+4.294	CRITICAL DEPTH
+8.00	+0.88	+2.03	+38.16	+4.199	CRITICAL DEPTH
+8.50	+0.68	+1.92	+35.43	+4.108	CRITICAL DEPTH
+9.00	+0.51	+1.81	+32.94	+4.022	CRITICAL DEPTH
+9.50	+0.36	+1.72	+30.68	+3.940	CRITICAL DEPTH
+10.00	+0.23	+1.63	+28.62	+3.863	CRITICAL DEPTH
+10.50	+0.13	+1.54	+26.75	+3.789	CRITICAL DEPTH
+11.00	+0.06	+1.47	+25.06	+3.719	CRITICAL DEPTH
+11.50	+0.01	+1.40	+23.52	+3.654	CRITICAL DEPTH
+12.00	+0.00	+1.33	+22.13	+3.592	CRITICAL DEPTH
+12.50	+0.01	+1.27	+20.86	+3.534	CRITICAL DEPTH

+13.00	+0.06	+1.21	+19.72	+3.480	CRITICAL DEPTH
+13.50	+0.13	+1.16	+18.79	+3.429	CRITICAL DEPTH
+14.00	+0.23	+1.11	+18.01	+3.380	CRITICAL DEPTH

Normal End of Program

Click here to view Table D-1. Routing of Tide Through a Constricted Bridge Opening - Md. Route 286 over Back Creek, Cecil County

Go to Appendix E

Appendix E : HEC 18

Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges

Go to Appendix F

This appendix contains relevant material for recording and coding the results of the evaluation of scour at bridges. The material is excerpted from the Federal Highway Administration document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," dated 1995.

Items 58 Through 62 - Indicate the Condition Ratings

In order to promote uniformity between bridge inspectors, these guidelines will be used to rate and code items 58, 59, 60, 61, and 62.

Conditions ratings are used to describe the existing, in-place bridge as compared to the as-built condition. Evaluation is for the materials related physical condition of the deck, superstructure, and substructure components of a bridge. The condition evaluation of channels and channel protection and culverts is also included. Condition codes are properly used when they provide an overall characterization of the general condition of the entire component being rated. Conversely, they are improperly used if they attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition code must, therefore, consider both the severity of the deterioration or disrepair and the intent to which it is widespread throughout the component being rated.

The load-carrying capacity will not be used in evaluating condition items. The fact that a bridge was designed for less than current legal loads and may be posted shall have no influence upon condition ratings.

Portions of bridges that are being supported or strengthened by temporary members will be rated based on their actual condition; that is, the temporary members are not considered in the ratings of the item. (see Item 103-Temporary Structure Designation for the definition of a temporary bridge.)

Completed bridges not yet opened to traffic, if rated, shall be coded as if open to traffic.

Item 60 - Substructure (1 Digit)

This item describes the physical condition of piers, abutments, piles, fenders, footings, or other components. Rate and code condition in accordance with the previously described general condition ratings. Code N for all culverts.

All substructure elements should be inspected for visible signs of distress including evidence of cracking, section loss, settlement, misalignment, scour, collision damage, and corrosion. The rating given by item 113 - Scour Critical Bridges, may have significant effect on Item 60 if scour has substantially affected the overall condition of the substructure.

The substructure condition rating shall be made independent of the deck and superstructure.

Integral-abutment wingwalls to the first construction or expansion joint shall be included in the elevation. For non-integral superstructure and substructure units, the substructure shall be considered as the portion below the bearings. For structures where the substructure and superstructure are integral, the substructure shall be considered as the portion below the superstructure.

The following general condition rating shall be used as a guide in evaluating items 58, 59, and 60:

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION - out-of-service - beyond corrective action

Item 61 - Channel and Channel Protection (1 Digit)

This item describes the physical conditions associated with the flow of water through the bridge such as stream stability and the condition of the channel. riprap, slope protection, or stream control devices including spur dikes. The inspector should be particularly concerned with visible signs of excessive water velocity which may affect undermining of slope protection, erosion of banks, and realignment of the stream which may result in immediate or potential problems. Accumulation of drift and debris on the superstructure and substructure should be noted on the inspection form but not included in the condition rating.

Rate and code the condition in accordance with the previously described general condition ratings and the following descriptive codes:

Code	Description
N	NOT APPLICABLE - Use when bridge is not over a waterway (channel).
9	There are no noticeable or noteworthy deficiencies which affect the condition of the channel.
8	Banks are protected or well vegetated. River control devices and embankment protection have a little minor damage. Banks and/or channel have minor amounts of drift.
7	Bank is beginning to slump. River control devices and embankment protection have little minor damage. Banks and/or channel have minor amounts of drift.
6	Bank is beginning to slump. River control devices and embankment protection have widespread minor damage. There is minor stream bed movement evident. Debris is restricting the channel slightly.
5	Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel.
4	Bank and embankment protection is severely undermined. River control devices have severe damage. Large deposits of debris are in the channel.
3	Bank protection has failed. River control devices have been destroyed. Stream bed aggradation, degradation or lateral movement has changed the channel to now threaten the bridge and/or approach roadway.
2	The channel has changed to the extent the bridge is near a state of collapse.
1	Bridge closed because of channel failure. Corrective action may put back in light service.
0	Bridge closed because of channel failure. Replacement Necessary.

Item 71 - Waterway Adequacy (1 Digit)

This item appraises the waterway opening with respect to passage of flow through the bridge. The following codes shall be used in evaluating waterway adequacy (interpolate where appropriate). Site conditions may warrant somewhat higher or lower ratings than indicated by the table (e.g., flooding of an urban area due to a restricted bridge opening).

Where overtopping frequency information is available, the description given in the table for chance of overtopping mean the following:

Remote	-	greater than 100 years
Slight	-	11 to 100 years
Occasional	-	3 to 10 years
Frequent	-	less than 3 years

Adjectives describing traffic delays mean the following:

Insignificant	-	Minor inconvenience. Highway passable in a matter of hours.
Significant	-	Traffic delays of up to several days.
Severe	-	Long term delays to traffic with resulting hardship.

Functional Classifications			Description
Principal Arterials Interstates Freeways, or Expressways	Other Principal and Minor Arterials and major collectors	Minor Collectors, Locals	
Code			
N	N	N	Bridge not over a waterway.

9	9	9	Bridge deck and roadway approaches above flood water elevation (high water). Chance of overtopping is remote.
8	8	8	Bridge deck above roadway approaches. Slight chance of overtopping roadway approaches.
6	6	7	Slight chance of overtopping bridge deck and roadway approaches.
4	5	6	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with insignificant traffic delays.
3	4	5	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with significant traffic delays.
2	3	4	Occasional overtopping of bridge deck and roadway approaches with significant traffic delays.
2	2	3	Frequent overtopping of bridge deck and roadway approaches with significant traffic delays.
2	2	2	Occasional or Frequent overtopping of bridge deck and roadway approaches with sever traffic delays.
0	0	0	Bridge closed.

Item 92 - Critical Feature Inspection (9 Digits)

Using a series of 3-digit code segments, denote critical features that need special inspections or special emphasis during inspections and the designated inspection interval in months as determined by the individual in charge of the inspection program. The designated inspection interval could vary from inspection to inspection depending on the condition of the bridge at the time of inspection.

Segment	Description	Length
92A	Fracture Critical Details	3 digits
92B	Underwater Inspection	3 digits
92C	Other Special Inspection	3 digits

For each segment of item 92A, B, and C, code the first digit Y for special inspection or emphasis needed and Code N for not needed. The first digit of item 92A, B, and C must be coded N for all structure to designate either a yes or no answer. Those bridges coded with a Y in item 92A or B should be the same bridges contained in the Master Lists of fracture critical and special underwater inspection bridges. In the second and third digits of each segment, code a 2-digit number to indicate the number of months between inspections only if the first digit is coded Y. If the first digit is coded N, the second and third digits are left blank.

Current guidelines for the maximum allowable interval between inspections can be summarized as follows:

Fracture Critical Details	24 months
Underwater Inspection	60 months
Other Special Inspections	24 months

Examples:	ITEM	CODE
A 2-girder system structure which is being inspected yearly and no other special inspections are required.	92A 92B 92C	Y12 N___ N___
A structure where both fracture critical and underwater inspection are being performed on a 1-year interval. Other special inspections are not required.	92A 92B 92C	Y12 Y12 N___
A structure has been temporarily shored and is being inspected on a 6-month interval. Other special inspections are not required.	92A 92B 92C	N___ N___ Y06

Item 93 - Critical Feature Inspection Date (12 digits)

Code only if the first digit of item 92A, B, or C is coded Y for yes. Record as a series of 4-digit code segments, the month and year that the last inspection of the denoted critical feature was performed.

Segment	Description	Length
93A	Fracture Critical Details	4 digits
93B	Underwater Inspection	4 digits
93C	Other Special Inspection	4 digits

For each segment of this item, when applicable, code a 4-digit number to represent the month and year. The number of the month should be coded in the first 2 digits with a leading zero as required and the last 2 digits of the year coded as the third and fourth digits of the field. If the first digit of any part of item 92 is coded N, then the corresponding part of this item shall be blank.

Examples:	ITEM	CODE
A structure has a fracture critical members which were last inspected in March 1986. It does not require under-water or other special feature inspections.	93A 93B 93C	0386 (blank) (blank)
A structure has no fracture critical details, but requires underwater inspection and has other special features (for example, a temporary support) for which the state requires special inspection. The last underwater inspection was done in April 1986 and the last special feature inspection was done in November 1985.	92A 92B 92C	(blank) 0486 1185

Item 113 - Scour Critical Bridges

Use a single-digit code as indicated below to identify the current status of the bridge regarding its vulnerability to scour. Scour analyses shall be made by hydraulic/geotechnical/structural engineers. Details on conducting a scour analysis are included in the FHWA Technical Advisory 5140.23 titled "Evaluating Scour at Bridges." Whenever a rating factor of 4 or below is determined for this item, the rating factor for item 60 - Substructure may need to be revised to reflect the severity of actual scour and resultant damage to the bridge. A scour critical bridge is one with abutment or pier foundations which are rated as unstable due to:

1. Observed scour at the bridge site
2. A scour potential as determined from a scour elevation study

Code	Description
N	Bridge is not over a waterway (channel)
U	Bridge with "unknown" foundation that has not been evaluated for scour. Since risk cannot be determined, flag for monitoring during flood events and, if appropriate, closure.
T	Bridge over "tidal" waters that has not been evaluated for scour, but considered low risk. Bridges will be monitored with regular inspection cycle and with appropriate underwater inspections. ("Unknown" foundations in "Tidal" waters should be coded U.)
9	Bridge foundations (including piles) on dry land well above flood water elevations.
8	Bridge foundations determined to be stable for accessed or calculated scour conditions; calculated scour is above top of footing. (Example A)
7	Countermeasures have been installed to correct a previously existing problem with scour. Bridge is no longer scour critical.
6	Scour calculation/evaluation has not been made (Use only to describe case where bridge has not yet been evaluated for scour potential.)
5	Bridge foundations determined to be stable for calculated scour conditions; Scour within limits of footing or piles. (Example B)
4	Bridge foundations determined to be stable for calculated scour conditions; field review indicates action is required to protect exposed foundations from effects of additional erosion and corrosion.

3 Bridge is scour critical; bridge foundations determined to be unstable for calculated scour conditions:

- Scour within limits of footing or piles. (Example B)
- Scour below spread-footing base or pile tips. (Example C)

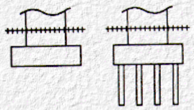
2 Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations. immediate action is required to provide scour countermeasures

1 Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic.

0 Bridge is scour critical. Bridge has failed and is closed to traffic.

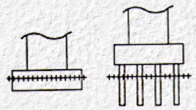
Examples:

A. Above top of footing



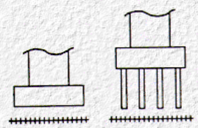
None - Indicate rating of 8 for this item

B.Within limits of footing or piles



Conduct foundation structural analysis

C. Below pile tips or spread-footing base



Provide for monitoring and scour counter-measures as necessary

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Go to Appendix F

Appendix F : HEC 18

Scour Measuring and Monitoring Equipment

Go to Appendix G

F-1 Introduction

As discussed in Chapter 7, scour monitoring is considered to be a suitable countermeasure for scour. Scour monitoring is differentiated from inspection in that monitoring generally implies the determination of the bed elevation at the time that scour is occurring. Although simple in concept, the ability to monitor scour during floods is inhibited by high flow depths and velocities, turbidity, floating debris, turbulence and ice. It is because of the adverse environment which exists in and around bridge piers and abutments during high flows (when maximum scour generally occurs) that, until recently, there were few instruments and techniques available to measure scour. Recent research has resulted in development of mobile and fixed instrumentation for measuring scour during flood flows.[78] This instrumentation has proven useful for monitoring scour and inspecting general streambed conditions for routine maintenance and scour evaluations.

Past techniques to measure scour have focused on manual mechanical methods such as using a graduated rod to probe the scour hole, using a cable and lead weight, or similar techniques. Sonic fathometers have also been used with varying degrees of success. In a few notable cases divers have attempted to probe the scour holes around bridge piers at high water, but these few attempts have proven to be extremely dangerous given the nature of the turbulence around a bridge foundation, particularly during flood flows.

Geophysical tools and techniques have also been adapted or are being developed to measure and monitor scour at bridge piers. Some of these techniques can be employed as post-flood inspection methods to determine maximum scour depths after floods.

The following text discusses some of the most promising techniques and instruments which are, or may be available in the future to monitor and measure scour at bridge piers and abutments. In the following paragraphs, techniques for either mobile or fixed installation scour monitoring devices are discussed. Then various geophysical tools which have been, or could be utilized for scour monitoring or post-flood inspection are described.

F-2 Mobile Instrumentation

Mobile instrumentation comprises all instrumentation which can be brought to a bridge site to measure scour at both flood and normal conditions. Typically, these instruments are deployed on a boat, unmanned floating equipment platforms, from the bridge, or other means to sense the bed along and around the bridge piers and abutments. In some cases, sonic transducers have been attached to sounding weights and suspended over the bridge rail using a portable crane and winch arrangement.

Mobile instrumentation can range from a simple black and white fathometer (typically used by fishermen) to ground penetrating radar, tuned transducers, color fathometers, or other geophysical techniques. Cable and a lead weight similar to that used for stream gaging are also used for scour measurement. More recently, two- and three-dimensional sonic fathometers which can produce three-dimensional images have become available.

An advantage of these techniques is that since the instrumentation is mobile, the equipment can be used to service many bridges within a highway department's region. This feature makes mobile instruments particularly useful to inspectors involved in the 2- and 5-year bridge inspection cycle. Many state DOTs have been using black and white fathometers for developing cross section surveys of the bridge waterway area as well as for scour monitoring.

Disadvantages of mobile instrumentation relate to the inherent dangers and difficulties involved in collecting data, particularly during flood flows. In addition, some of the instrumentation requires technically qualified personnel to operate and maintain the device and interpret data.

F-3 Fixed Instrumentation

Scour monitoring equipment can be deployed in a fixed installation mode to provide a scour monitoring capability. In a typical installation, an instrument combined with a method to either manually or digitally record scour data can be installed on or near a bridge pier or abutment to provide scour monitoring or measuring. These instruments include low-cost or more sophisticated sonic fathometers, sounding rods, or buried or driven rods.

Due to the wide variety of pier and abutment geometries, and because of the variability in river geometry, flow conditions, bed material, and other characteristics of highway crossings, no single fixed instrumentation type will be applicable to meet the needs of all cases. Rather, there is a need to have a variety of fixed instrumentation to meet the needs of the varied conditions found at bridges.

F-3.1 Sounding Rods

In the context of fixed scour monitoring equipment, the use of sounding rods encompasses methods whereby a rod resting on the bed is allowed to slide vertically as scour develops. The rod is constrained to vertical movement as scour develops by means of a sleeve or other method which will orient the sounding rod directly above the scour hole but will allow the rod to move vertically. Scour depths can be determined either manually or by using data logging techniques. One such instrument, known as the Brisco Monitor (use of trade names is for identification purposes only), is currently commercially available. This instrument measures scour by measuring the length of cable, which is attached to the top of the sounding rod, unwound from a spool in the data recording enclosure.

Sounding rods, such as the one described above, can be used as scour monitoring devices, however these instruments are limited by the expected ultimate depth of scour, and subsequently, the length of rod required to accurately track the development of scour. As the rod length increases, the weight of the rod bearing on the bed material also increases. The entire weight of the rod must be supported by the bed material of the scour hole. A footplate attached to the end of the sounding rod must be of sufficient size to prevent the rod from penetrating the bed.

F-3.2 Sonic Fathometers

Sonic fathometers can be attached to the bridge pier or abutment to monitor scour. Currently there are several research organizations experimenting with and field testing these types of instrumentation. For example, the USGS has instrumented several bridges using both a simple "fish finder" fathometers and more sophisticated commercial sonic fathometers. The Virginia Transportation Research Council has installed multiple transducers on a bridge south of Richmond, VA. This installation is equipped with data logging and telemetering capability. Under an NCHRP project (Project 21-3) to develop scour instrumentation, Ayres Associates (formerly Resource Consultants & Engineers, Fort Collins, Colorado) have reported successful operation of a "fish finder" type sonic fathometer linked to a datalogger at several tidal and riverine bridges.(78)

The bridge deck serviceable sonic fathometer developed under NCHRP Project 21-3 uses a low-cost commercial sonic fathometer to monitor and measure scour. The transducer for this device is mounted in a housing which is designed to slide freely inside a conduit mounted to the bridge pier or abutment and aimed at the location where scour is anticipated to occur (see Figure F-1). The transducer is inserted from the bridge deck into the conduit and snapped into place in the end of the conduit. This design allows for the servicing or replacement of the transducer without the need for scaffolds, inspection cranes, or divers.

Figure F-1. Sonic Instrument on Johns Pass Bridge, Florida

Figure F-2. Sonic Datalogger on Johns Pass Bridge, Florida

Scour data are obtained and recorded by a specially designed datalogger (Figure F-2) connected to the sonic fathometer. In this way, a time-stamped record of the scour and fill process can be recorded, stored, and retrieved for future analysis. Electrical power is provided by a battery and solar panel. This system also lends itself to telemetry of scour data from a remote site.

During field tests conducted by RCE from 1992-1995, this instrument performed well, documenting the scour and fill process. Installations have included bridges on the South Platte River in Colorado, the Rio Grande in New Mexico, as well as bridges in Texas and New York. The most challenging installation was on the Johns Pass bridge at a deep (14 m), aggressive tidal inlet on Florida's Gulf Coast. This instrument has proven to be extremely reliable in a very hostile tidal environment over a two-year period. The only significant limitation of this low-cost sonic system is its vulnerability to blockage when ice or debris accumulates on the bridge pier below the transducer. High velocity, sediment laden flows have not proven to be a limitation in deploying this device.

F-3.3 Buried or Driven ROB Instrumentation

This class of devices encompasses all instrumentation which could be mounted in or attached to a vertical support which is either buried or driven into the channel bed at the location where scour is expected to occur. By sensing the channel bed/water interface, the progression of scour can be monitored or measured.

Under NCHRP Project 21-3, a simple magnetic sliding collar was tested in the laboratory and deployed to field sites in response to the need to develop a simple device that might progress through the development stages quickly. This device consists of an open collar, which is free to slide over a small-diameter stainless steel pipe. A large bar magnet is mounted on the collar (Figure F-3). The pipe can be driven into the streambed at a location where scour is expected to occur. The pipe (which can be made and shipped in sections) is driven into the bed using either manual, pneumatic, or other mechanical methods. Scour is determined by inserting a magnetic sensor (probe) mounted at the end of a graduated cable into the support pipe or extension conduit from the bridge deck. When the sensor is close to the magnet on the collar, an audible signal (buzzer) is heard at the bridge deck.

Magnetic sliding collar devices have been installed in the field at many locations since 1992, including Colorado, Minnesota, Michigan, New Mexico, and Texas (see Figure F-4). Test sites have included a variety of geomorphic and geologic conditions and have exposed the instruments to impact from both ice and debris. The instrument is best suited to smaller bridges and shallower flow depths (3-5 m). Of the eight instruments tested in the field, only one has been destroyed by debris impact.

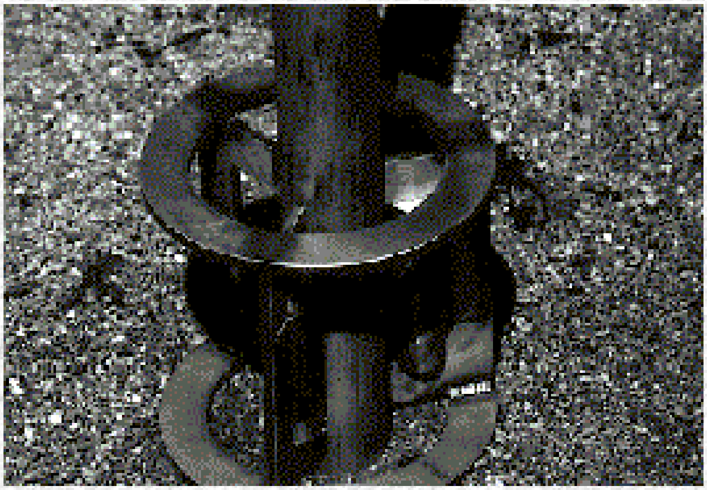

Figure F-3. Sliding Collar Instrument

Figure F-4. Instrument Installation, Michigan DOT

The design of this instrument is simple and it is easy to install and operate. The device can be fabricated in a variety of lengths and can be installed using bends to route the extension conduit from the top of the stainless steel pipe to the bridge deck.

Thus, the instrument can be fastened closely to most bridge piers and abutments to protect the instrument from debris and ice impact. In sand-bed streams, the instrument can be driven into the bed using a modified fence post-type driver. In more cohesive streambeds, the instrument can be installed by using a pneumatic post driver. At this time, only manual determination of the scour depth can be made using the probe and buzzer. However, a multiple magnetic sensor array, which can be used to upgrade existing field sites, has been designed and will be tested in the near future. This multiple sensor array allows linking the instrument to a datalogger so that the time history of scour can be automatically logged. It will also permit installing the instrument flush with the streambed, eliminating the vulnerable section of the extension conduit on the face of the bridge pier.

F-4 Geophysical Tools

After a flood, the stream velocity decreases which may result in the sediment being redeposited in the scour hole, (referred to as infilling). Since this material often has a different density than the adjacent unscoured material, the true extent of scour can be measured by determining the interface where the density change occurs. Methods for determining this interface includes standard penetration testing, cone penetrometer exploration, and geophysical techniques. While standard penetration testing is accurate it is expensive, time consuming and does not provide a continuous profile. Less expensive geophysical methods are available, which will provide continuous subsurface profiles by providing information on the physical properties of the substrate.

The three geophysical tools which have been used to measure scour after infilling occurs are: ground penetrating radar, tuned transducer, and color fathometer. Each of these methods has its advantages and limitations. However, if applied properly, they can yield meaningful data in a very short period of time. The U.S. Geological Survey in cooperation with the Federal Highway Administration has used each of these tools to study the extent of scour and the findings are documented in a report entitled "The Use of Surface Geophysical Methods in Studying River Bed Scour." The following descriptions are taken from that report by S.R. Gorin and F.P. Haeni of the U.S. Geological Survey.

F-4.1. Ground Penetrating Radar

Ground penetrating radar (GPR) can be used to obtain high resolution, continuous, subsurface profiles on land or in relatively shallow water (less than 7.6 m). This device transmits short, 80 to 800 MHz electromagnetic pulses into the subsurface and measures the two way travel time for the signal to return to the subsurface and measures the two way travel time for the signal to return to the receiver. When the electromagnetic energy reaches an interface between two materials with differing physical properties, a portion of the energy is reflected back to the surface, while some of it is attenuated and a portion is transmitted to deeper layers. The penetration depth of GPR is dependent upon the electrical properties of the material through which the signal is transmitted and the frequency of the signal transmitted. Highly conductive (low resistivity) materials such as clay materials severely attenuate radar signals. Similarly, sediments saturated with or overlain by salt water will yield poor radar results. Fresh water also attenuates the radar signal and limits the use of radar to sites with less than 7.6 m of water. The lower frequency signals yield better penetration and reduced resolution, whereas higher frequency signals yield higher resolution and less penetration. Ground penetrating radar systems which include a transmitter, receiver, and antenna and a high density tape recorder and player for storage of records cost approximately $50,000.

Figure F-5 shows a cross section generated by a ground penetrating radar signal upstream of a bridge pier. The scour hole is approximately 2.1 m deeper than the river bottom base level and 18 to 21 m wide. Two different infilled layers can be observed at this location. The apparent thickness of the infilled material at the center of the hole is 0.9 m to the first interface and 1.8 m to the second interface.

F-4.2 Tuned Transducer

The tuned transducer and the color fathometer are both seismic systems which operate through the transmission and reception of acoustic waves. A portion of the seismic signal is reflected back to the surface when there is a change in acoustical impedance between two layers. The major variable which separates these two devices from the standard fathometer is the frequency. The tuned transducer and color fathometer have lower frequency signals (20 KHz) which yield better penetration at the expense of resolution. High frequency fathometers (200 KHz) have good resolution with little or no penetration. In fine grained materials, up to 30.5 m of penetration can be obtained with a 3 to 7 KHz transducer, while in coarser material subsurface penetration may be limited to a meter. The tuned transducer system cost approximately $25,000.

Figure F-5 shows a cross section record provided by a 14 KHz tuned transducer. This is the same location as the GPR record in Figure F-4. The record shows 1.8 m of infilled material. The two layers which could be seen on the radar record are not evident on the tuned transducer record.

F-4.3 Color Fathometer

The color fathometer is a variable frequency seismic system that digitizes the reflected signal and displays a color image on a monitor. This system measures the reflected signal in decibels and it distinguishes between different interfaces by assigning color changes to a given degree of decibel change. Since decibel changes in the reflected signal are related to density, porosity and median grain size, the instrument is able to identify and define shallow interfaces in the subsurface. Where infilling has occurred, the soft material is easily penetrated and shown to have low reflectivity as opposed to denser materials which have high reflectivity. Typically, the materials which have a low reflectivity are assigned the "cool" colors such as blue and green while the denser material is represented by the "hot" colors such as red and orange. Since the data is displayed on a color monitor, a hard copy is not readily available; however, it can be stored on a cassette tape for playback and processing. The U.S. Geological Survey is presently working on developing a computer program to process the color fathometer record in order to remove some of the extraneous and undesirable signals which make interpretation more difficult.

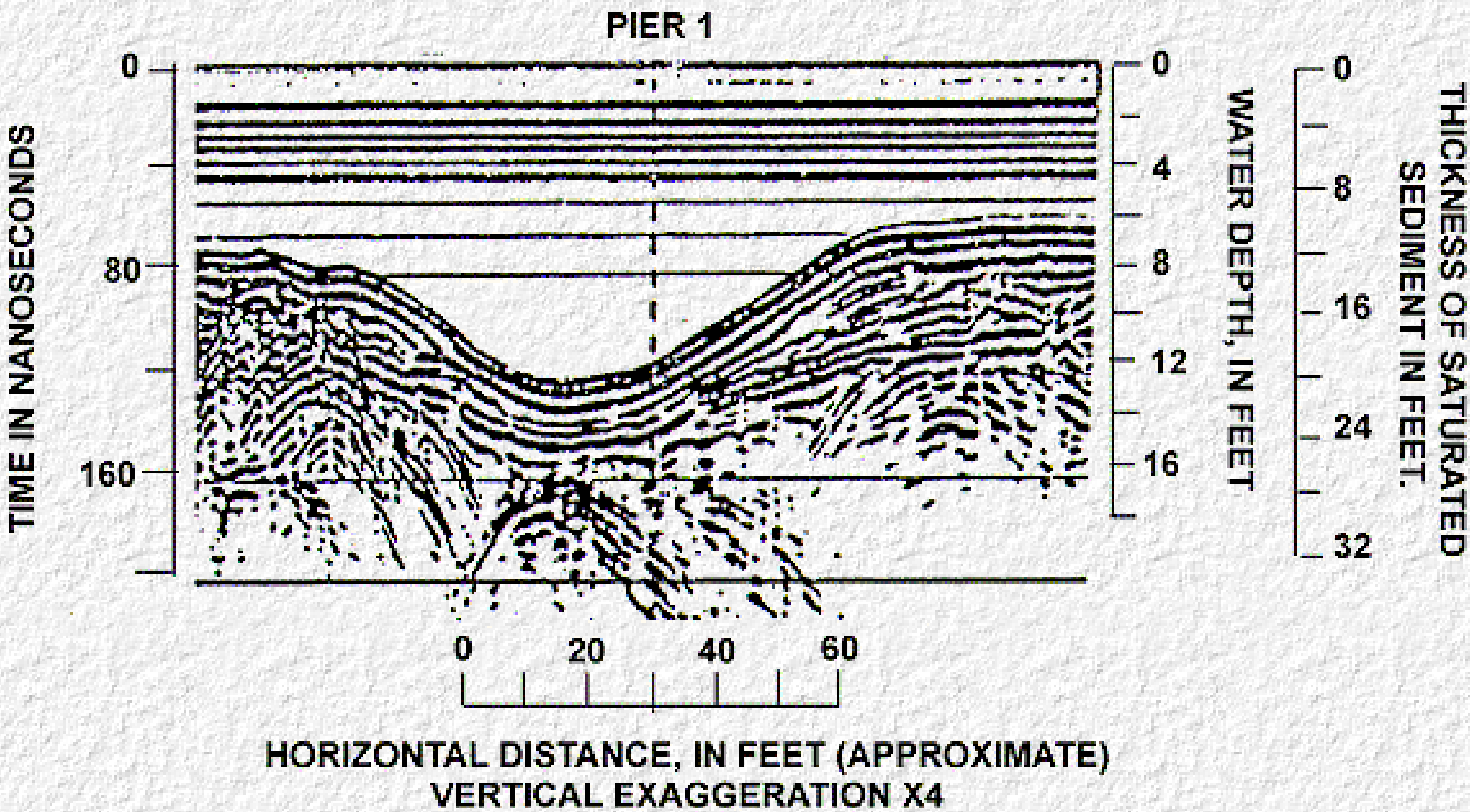

Figure F-5. Example of Ground Penetrating Radon

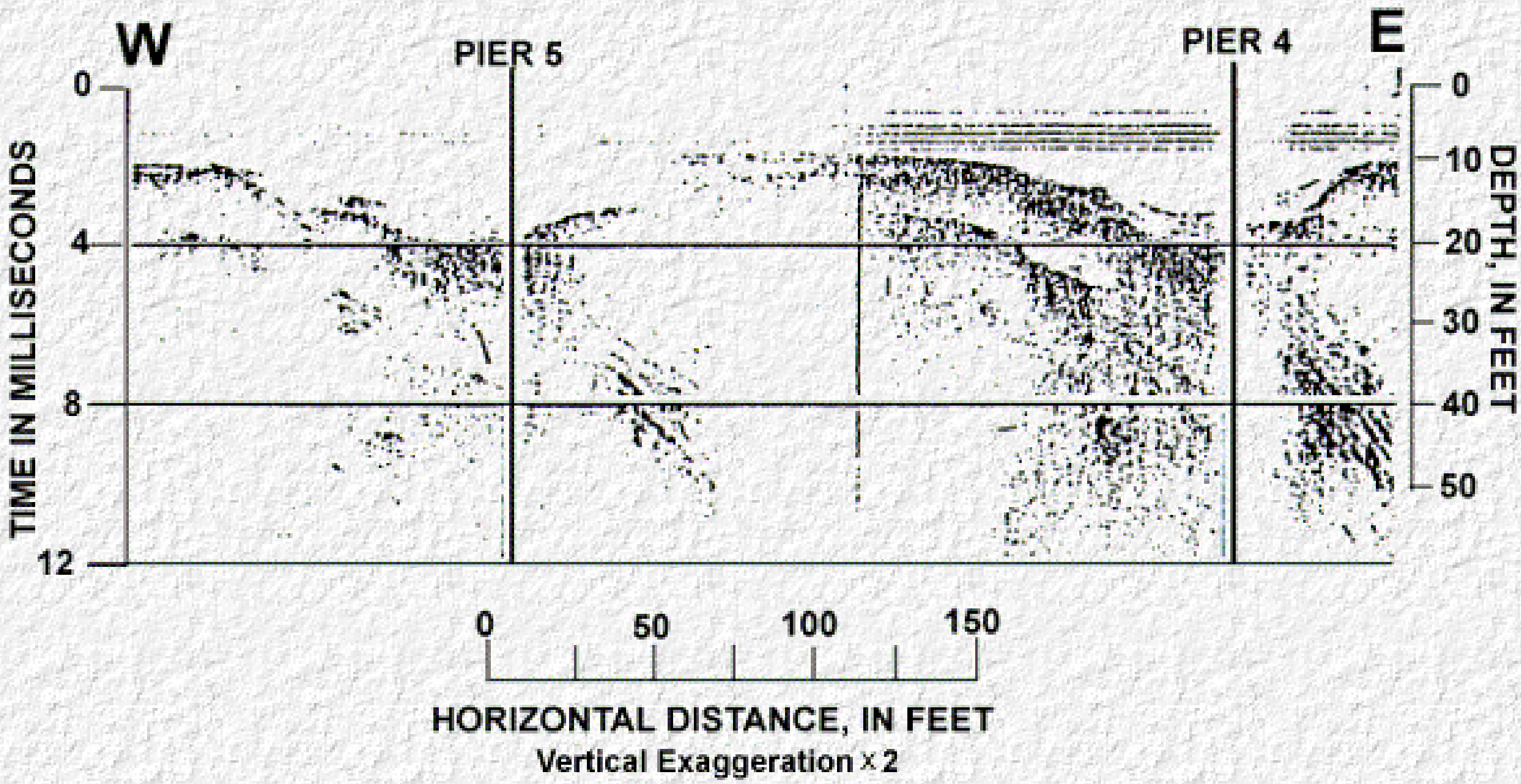

Figure F-6. Example of 14 KHz Tuned Transducer

F-4.4 Black and White Fathometer

Even though the black and white fathometer is unable to penetrate the channel except in very soft mud, it is still considered an excellent tool for defining the channel bottom. The graphic recorder is easy to use, reasonably inexpensive and will provide an accurate bottom profile very quickly. Also when used in conjunction with the other tools, it adds a degree of certainty to the other geophysical data. A 200 KHz fathometer with graphics capabilities can be purchased for approximately $1,000.

Figure F-7 shows a cross section using a 200 KHz fathometer. This record correlates with the radar and tuned transducer record shown in Figure F-5 and Figure F-6 with the exception that the radar record was run 1.8 m further upstream.

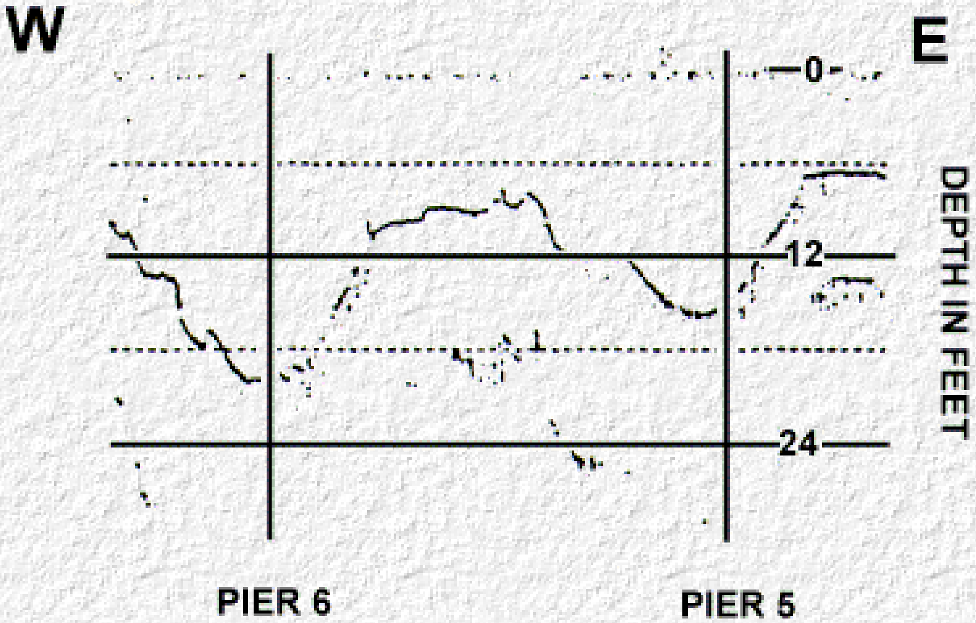

Figure F-7. Example of 200 KHz fathometer

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Appendix G : HEC 18

Interim Procedure for Existing Pier Scour with Debris

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G-1 Assumptions:

1. Debris aligns with the flow direction and attaches to the upstream nose of a pier. The width of accumulation, W, on each side of the pier is normal to the flow direction.

2. The trailing end of a long slender pier does not add significantly to pier scour for that portion of the length beyond 12 pier widths. This is consistent with the current guidelines in HEC 18 to cut K_2 at L/a=12

3. The affect of the debris in increasing scour depths is taken into account by adding a width, W, to the sides and front of the pier. Engineering judgement and experience is used to determine the width, W.

G-2 Suggested Procedure:

1. Use K_1 & K_2 = 1.0

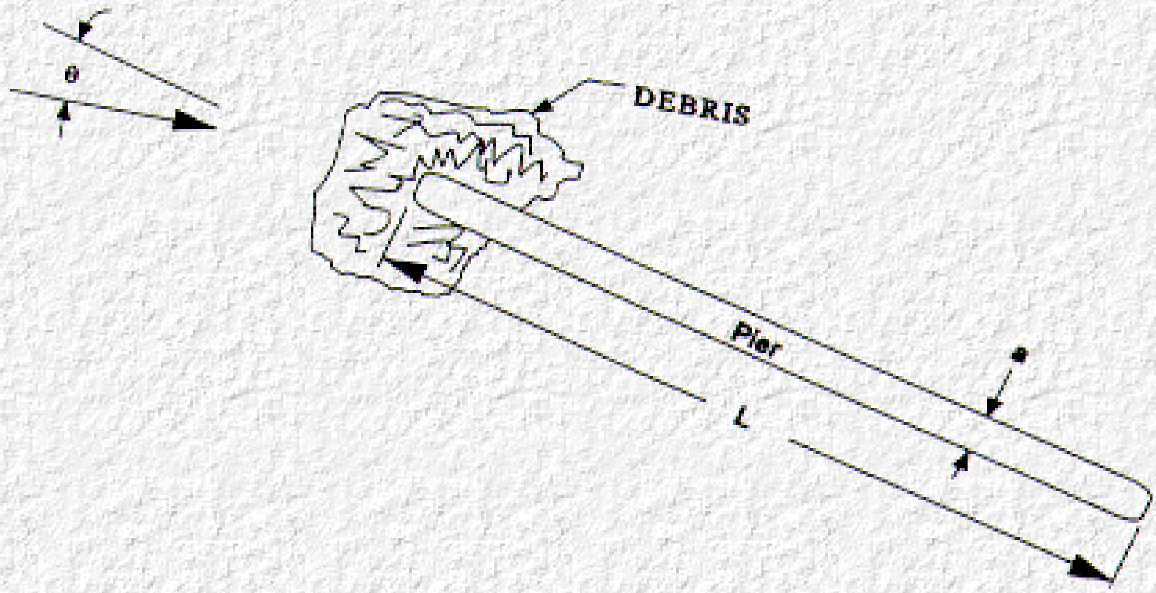
2. Project the debris pile up to twelve pier widths of the pier length normal to the flow direction as follows:

   L' = L or 12*a (whichever is less)

   $a_{proj} = 2W + a\cos\theta$ or $W + a\cos\theta + L'\sin\theta$ (whichever is greater)

3. Use K_1, K_2, K_3, K_4, & a_{proj} in the HEC 18 pier scour equation as follows:

$$\frac{y_s}{y_1} = 2.0(1.0)(1.0)K_3 K_4 \left(\frac{a_{proj}}{y_1}\right)^{0.65} Fr_1^{0.43}$$



G-3 Example:

NVFAS 228 Bridge over the Humbolt River South Fork
Flow: depth, y_1=2.42 m; V_1=3.60 m/s; Fr_1=0.74
Pier: a =0.46 m; L=5.49 m; Skew to flow direction=15 degrees.
Debris: Local assumption for accumulation W=0.61 m extended in front and on each side of pier.

Computations:

L/a=12.62/0.46=27.6>12: use L'=12*0.46=5.52 m

a_{proj} =1.22+0.46*cos 15^o=1.66 m or
0.61+0.46*cos 15^o+5.52*sin 15^o =2.48 m

use 2.48 m

$$\frac{y_s}{2.42} = 2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{2.48}{2.42}\right)^{0.65}(0.74)^{0.43}$$

$$y_s = 1.98 * 2.42 = 4.79 \text{ m}$$

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Chapter 1 : HEC 18
Introduction

Go to Chapter 2

1.1 Purpose

1. Designing new and replacement bridges to resist scour,
2. Evaluating existing bridges for vulnerability to scour,
3. Inspecting bridges for scour,
4. Providing scour countermeasures, and
5. Improving the state-of-practice of estimating scour at bridges.

1.2 Organization of This Circular

The procedures presented in this document contain the state-of-knowledge and practice for dealing with scour at highway bridges. Chapter 1 gives the background of the problem and general state-of-knowledge of scour. Basic concepts and definitions are presented in Chapter 2. Chapter 3 gives recommendations for designing bridges to resist scour. Chapter 4 gives equations for calculating and evaluating total scour depths at piers and abutments for both riverine and tidal waterways. Chapter 5 provides procedures for conducting scour evaluations and analyses at existing bridges. Chapter 6 presents guidelines for inspecting bridges for scour. Chapter 7 gives a plan of action for installing countermeasures to strengthen bridges that are considered vulnerable to scour.

This edition of HEC 18 uses SI **metric** units. **In Appendix A, the metric (SI) unit of measurement is explained. The conversion factors, physical properties of water in SI system of units, sediment particle size grade scale, and some common equivalent hydraulic units are also given.** This edition uses for the unit of length the meter (m), of mass the kilogram (kg), of weight/force the newton (N), of pressure the Pascal (Pa, N/m^2), and of temperature the degree centigrade (°C). The unit of time is the same in SI as in English system (seconds, s). Sediment particle size is given in millimeters (mm), but in calculations the decimal equivalent of millimeters in meters is used (1 mm = 0.001 m). The value of some hydraulic engineering terms used in the text in SI units and their equivalent English units are given in Table 1.

Table 1. Commonly Used Engineering Terms in SI and English Units

Term	SI Units	English Units
Length	1 m	3.28 ft
Volume	1 m^3	35.31 ft^3

Discharge	1 m^3/s	35.31 ft^3/s
Acceleration of Gravity	9.81 m/s^2	32.2 ft/s^2
Unit Weight of Water	9800 N/m^3	62.4 $lb./ft^3$
Density of Water	1000 kg/m^3	1.94 $slugs/ft^3$
Density of Quartz	2647 kg/m^3	5.14 $slugs/ft^3$
Specific Gravity of Quartz	2.65	2.65
Specific Gravity of Water	1	1
Temperature	°C = 5/9 (°F - 32)	°F

1.3 Background

The most common cause of bridge failures is floods with the scouring of bridge foundations being the most common cause of flood damage to bridges. **The hydraulic design of bridge waterways is typically based on flood frequencies somewhat less than those recommended for scour analysis in this publication.** During the spring floods of 1987, 17 bridges in New York and New England were damaged or destroyed by scour. In 1985, 73 bridges were destroyed by floods in Pennsylvania, Virginia, and West Virginia. A 1973 national study for the FHWA of 383 bridge failures caused by catastrophic floods showed that 25 percent involved pier damage and 72 percent involved abutment damage.(1) A second more extensive study in 1978 indicated local scour at bridge piers to be a problem about equal to abutment scour problems.(2) A number of case histories on the causes and consequences of scour at major bridges are presented in Transportation Research Record 950.(3)

From available information, the 1993 flood in the upper Mississippi basin, caused 23 bridge failures for an estimated damage of $15 million. The modes of bridge failures were 14 from abutment scour, 2 from pier scour only, 3 from pier and abutment scour, 2 from lateral bank migration, 1 from debris load, and 1 from unknown scour.(4)

In the 1994 flooding from storm Alberto in Georgia, there were over 500 state and locally owned bridges with damage attributed to scour. Thirty-one of state-owned bridges experienced from 15 to 20 feet of contraction scour and/or long-term degradation in addition to local scour. These bridges had to be replaced. Of more than 150 bridges identified as scour damaged, the State also recommended that 73 non-federal aid bridges be repaired or replaced. Total damage to the highway system was approximately $130 million.(4)

The American Association of State Highway and Transportation Officials (AASHTO) standard specifications for highway bridges has the following requirements to address the problem of stream stability and scour.(86)

- Hydraulic studies are a necessary part of the preliminary design of a bridge and should include . . . estimated scour depths at piers and abutments of proposed structures.

- The probable depth of scour shall be determined by subsurface exploration and hydraulic studies. Refer to Article 1.3.2 and FHWA (HEC 18) for general guidance regarding hydraulic studies and design.

- . . . in all cases, the pile length shall be determined such that the design structural load may be safely supported entirely below the probable scour depth.

1.4 Objectives of a Bridge Scour Evaluation Program

The need to minimize future flood damage to the nation's bridges requires that additional attention be devoted to developing and implementing improved procedures for designing and inspecting bridges for scour.(5) Approximately 84 percent of the 575 000 bridges in the National Bridge Inventory are built over waterways. Statistically, we can expect hundreds of these bridges to experience floods in the magnitude of a 100-year flood or greater each year. Because it is not economically feasible to construct all bridges to resist all conceivable floods, or to install scour countermeasures at all existing bridges to ensure absolute invulnerability from scour damage, some risks of failure from future floods may have to be accepted. **However, every bridge over water, whether existing or under design, should be assessed as to its vulnerability to floods in order to determine the prudent measures to be taken.** The added cost of making a bridge less vulnerable to scour is small when compared to the total cost of a failure which can easily be two to ten times the cost of the bridge itself. Moreover, the need to ensure public safety and minimize the adverse effects resulting from bridge closures requires our best efforts to improve the state-of-practice for designing and maintaining bridge foundations to resist the effects of scour.

The procedures presented in this manual serve as guidance for implementing the recommendations contained in the FHWA Technical Advisory T5140.23 entitled, "Evaluating Scour at Bridges."(6) The recommendations have been developed to summarize the essential elements which should be addressed in developing a comprehensive scour evaluation program. A key element of the program is the identification of scour-critical bridges which will be entered into the National Bridge Inventory using the FHWA document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges."(7)

1.5 Improving the State-of-Practice of Estimating Scour at Bridges

Some of the problems associated with estimating scour and providing cost-effective and safe designs are being addressed in research and development programs of the FHWA and individual state highway agencies. The following sections detail the most pressing research needs.

1. **Field Measurements of Scour**. The current equations and methods for estimating scour at bridges are based primarily on laboratory research. Very little field data have been collected to verify the applicability and accuracy of the various design procedures for the range of soil conditions, streamflow conditions, and bridge designs encountered throughout the United States. In particular, states are

encouraged to initiate studies for the purpose of obtaining field measurements of scour and related hydraulic conditions at bridges for evaluating, verifying, and improving existing scour prediction methods. In excess of 20 states have initiated cooperative studies with the Water Resources Division of the U.S. Geological Survey (USGS) to collect scour data at existing bridges. A model cooperative agreement with the USGS for purposes of conducting a scour study was included in the FHWA guidance "Interim Procedures for Evaluating Scour at Bridges," which accompanied the September 1988 FHWA Technical Advisory.(8,6)

2. **Scour Monitoring and Measurement Equipment**. There is a need for the development of instrumentation and equipment to indicate when a bridge is in danger of collapsing due to scour. Many bridges in the United States were constructed prior to the development of scour estimation procedures. Some of these bridges have foundations which are vulnerable to scour; however, it is not economically feasible to repair or replace all of these bridges. Therefore, these bridges need to be monitored during floods and closed before they fail.

The FHWA, in cooperation with state highway agencies and the Transportation Research Board, has initiated research to develop scour monitoring and measuring instruments. This research has developed several instruments for scour monitoring and measurement (see Appendix F). Research has also been initiated to develop techniques and instruments to identify the type and depth of unknown foundations for existing bridges.

3. **Scour Analysis Software**. There is a continued need for the development and maintenance of computer software for the analysis of all aspects of bridge scour. The FHWA microcomputer software WSPRO is recommended for obtaining hydraulic variables for scour computations. A software program to determine total scour at bridge crossings has been developed (BRI-STARS) and is under contract for improvement by FHWA. This software is available from McTrans Center, 512 Weil Hall, University of Florida, Gainsville, Florida 32611-2083 or PC Trans, University of Kansas, Transportation Center, 2011 Learned Hall, Lawrence, Kansas 66045-2962. A pooled fund research project is underway to adapt existing one-dimensional and two-dimensional computer models to determine the hydraulic variables to use in the scour equations for tidal streams.(9)

4. **Field and Laboratory Studies of Scour**. Laboratory studies are needed to better understand certain elements of the scour processes and develop alternate and improved scour countermeasures. Only through controlled experiments can the effect of the variables and parameters associated with scour be determined. Through these efforts, scour prediction equations can be improved and additional design methods for countermeasures developed. Results from these laboratory experiments must be verified by ongoing field measurements of scour.

Laboratory research is needed to:

- Improve methods to predict scour depths associated with pressure flow,

B. Determine more applicable equations for abutment scour to replace equations that inappropriately use abutment length, as a primary factor,

C. Improve methods for estimating scour when abutments are set back from the channel with overbank flow,

D. Conduct fundamental research on the mechanics of riverine and tidal scour,

E. Determine methods to predict scour depths when there is ice or debris buildup at a pier or abutment,

F. Improve our knowledge of the influence of graded, armored, or cohesive bed material on maximum local scour at piers and abutments,

G. Improve equations for determining scour depths of pile caps or footings located at different elevations in the flow or soil,

H. Improve methods for determining the size and placement (elevation, width, and location) of riprap in the scour hole to protect piers and abutments,

I. Determine the width of scour hole as a function of scour depth and bed material size,

J. Improve our knowledge of the effects of flow depth and velocity on scour depths,

K. Improve our understanding of the bridge scour failure mechanism which would combine the various scour components (pier, abutment, contraction, lateral migration, degradation) into an estimate of the scoured cross section under the bridge,

L. Improve methods to predict the effect of flow angle of attack against a pier or abutment on scour depth,

M. Determine the effect of wide piers and variable pier widths on scour depths,

N. Determine the impact of overlapping scour holes, and

O. Determine scour depths in structures designed as bottomless culverts, that is culverts founded on spread footings and placed on erodible soil.

Go to Chapter 2

Table 15. Assessing the Scour Potential at Bridges.

1. <u>UPSTREAM CONDITIONS</u>

    . <u>Banks</u>

    <u>STABLE</u>: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions; channel stabilization measures such as dikes and jetties.

    <u>UNSTABLE</u>: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization measures etc.

    b. <u>Main Channel</u>

    - Clear and open with good approach flow conditions, or meandering or braided with main channel at an angle to the orientation of the bridge.
    - Existence of islands, bars, debris, cattle guards, fences that may affect flow.
    - Aggrading or degrading streambed.
    - Evidence of movement of channel with respect to bridge (make sketches, take pictures).
    - Evidence of ponding of flow.

    . <u>Floodplain</u>

    - Evidence of significant flow on floodplain.
    - Floodplain flow patterns - does flow overtop road and/or return to main channel?
    - Existence and hydraulic adequacy of relief bridges (if relief bridges are obstructed, they will affect flow patterns at the main channel bridge).
    - Extent of floodplain development and any obstruction to flows approaching the bridge and its approaches.
    - Evidence of overtopping approach roads (debris, erosion of embankment slopes, damage to riprap or pavement, etc.).
    - Evidence of ponding of flow.

    . <u>Debris</u>

    - Extent of debris in upstream channel.

    e. <u>Other Features</u>

    - Existence of upstream tributaries, bridges, dams, or other features, that may affect flow conditions at bridges.

2. <u>CONDITIONS AT BRIDGE</u>

    . <u>Substructure</u>

    - Is there evidence of scour at piers?
    - Is there evidence of scour at abutments (upstream or downstream sections)?
    - Is there evidence of scour at the approach roadway (upstream or downstream)?

- Are piles, pile caps or footings exposed?

- Is there debris on the piers or abutments?

- If riprap has been placed around piers or abutments, is it still in place?

. Superstructure

- Evidence of overtopping by floodwater (Is superstructure tied down to substructure to prevent displacement during floods?)

- Obstruction to flood flows (Does superstructure collect debris or present a large surface to the flow?)

- Design (Is superstructure vulnerable to collapse in the event of foundation movement, e.g., simple spans and nonredundant design for load transfer?)

c. Channel Protection and Scour Countermeasures

- Riprap (Is riprap adequately toed into the streambed or is it being undermined and washed away? Is riprap pier protection intact, or has riprap been removed and replaced by bed-load material? Can displaced riprap be seen in streambed below bridge?)

- Guide banks (Spur dikes) (Are guide banks in place? Have they been damaged by scour and erosion?)

- Stream and streambed (Is main current impinging upon piers and abutments at an angle? Is there evidence of scour and erosion of streambed and banks, especially adjacent to piers and abutments? Has stream cross section changed since last measurement? In what way?)

d. Waterway Area Does waterway area appear small in relation to the stream and floodplain? Is there evidence of scour across a large portion of the streambed at the bridge? Do bars, islands, vegetation, and debris constrict the flow and concentrate it in one section of the bridge or cause it to attack piers and abutments? Do the superstructure, piers, abutments, and fences, etc., collect debris and constrict flow? Are approach roads regularly overtopped? If waterway opening is inadequate, does this increase the scour potential at bridge foundations?

3. DOWNSTREAM CONDITIONS

. Banks

STABLE: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions, channel stabilization measures such as dikes and jetties.

UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization measures, etc.

b. Main Channel

- Clear and open with good "getaway" conditions, or meandering or braided with bends, islands, bars, cattle guards, debris, and fences that retard and obstruct flow.

- Aggrading or degrading streambed.

- Evidence of movement of channel with respect to the bridge (make sketches and take pictures).

- Evidence of extensive bed erosion.

. Floodplain

- Clear and open so that contracted flow at bridge will return smoothly to floodplain, or restricted and blocked by dikes, development, trees, debris, or other obstructions.

- Evidence of scour and erosion due to downstream turbulence.

. Other Features

- Downstream dams or confluence with larger stream which may cause variable tailwater depths. (This may create conditions for high velocity flow through bridge.)

Table D-1. Routing of Tide Through a Constricted Bridge Opening - Md. Route 286 over Back Creek, Cecil County

A	C	D	F	G	H	I	J	L	M	N	O	Q	R	S	T	U	V
Time	Tidal El.	Waterway	El. Basin	Surf. Area	Ave. Tide	Ave. Area	Critical	Ave. Basin	Ave. Surf	Delta Hs	Hs-Ht	Velocity	Discharge	Velocity	Discharge	Sum of ft.	Sum of ft.
	Ht	Ac	Hs	As	Ht (avg)	Ac (avg)	Flow Area	Hs (avg)	As (avg)	Hs1-Hs2	Cl.K-Cl.H	V orifice	Q orifice	V critical	Q critical	Side: Eq3	Side: Eq3
(MIN)	(M)	(M~2)	(M)	(M^2.10^6)	(M)	(M^2)	M^2	(M)	(M^2/10^60	(M)	(M)	(M/S)	(CMS)	(M/S)	(CMS)	(CMS)	(CMS)
270	2.420 6	11.97	2.438	0.948 285 0													
285	2.313 5	11.97	2.438	0.948 285 0	2.367 11	11.97	10.531 16	2.438	0.948 285 0	0	0.070 887	1.179 3	14.116 53	4.465 8	47.050 69	18	14.116 5
300	2.204 0	11.97	2.438	0.947 584 5	2.588 0	11.97	10.523 04	2.438	0.947 934 7	0.004	0.177 190	1.864 5	22.318 45	4.465 8	48.978 89	22.213 04	22.318 4
315	2.092 6	11.97	2.438	0.945 830 6	2.148 32	11.97	10.494 60	2.249	0.946 707 6	0.01	0.280 670	2.346 6	28.089 37	4.459 2	46.798 18	28.518 97	28.089 34
330	1.979 6	11.97	2.41	0.943 368 5	2.036 14	11.97	10.445 86	2.417	0.944 599 6	0.014	0.380 854	2.733 5	32.720 74	4.450 4	46.488 78	32.693 77	32.720 7
345	1.865 7	11.97	2.392	0.940 191 3	1.922 79	11.97	10.380 87	2.401	0.914 779 9	0.018	0.478 262	3.063 2	36.667 11	4.438 6	46.077 26	36.835 59	36.667 1
360	1.751 3	11.97	2.371	0.936 467 9	1.808 59	11.97	10.301 66	2.381 5	0.938 329 6	0.021	0.572 909	3.352 6	40.131 60	4.424 2	45.577 38	39.894 35	40.131 60
375	1.636 9	11.834 28	2.347	0.932 190 3	1.694 19	11.902 1	10.210 26	2.359	0.934 329 1	0.024	0.664 806	3.611 5	42.985 53	4.407 6	44.379 91	42.915 44	42.985 51
390	1.523 0	11.140 53	2.322	0.927 708 8	1.580 03	11.487 4	10.110 74	2.334 5	0.929 949 8	0.025	0.754 465	3.847 4	44.196 84	4.389 3	43.747 28	43.831 93	44.196 8
405	1.410 1	10.452 68	2.297	0.923 200 8	1.466 60	10.796 6	10.009 19	2.309 5	0.925 454 8	0.025	0.842 897	4.066 6	43.906 03	4.370 7	43.117 80	43.707 07	43.906 0
420	1.298 6	9.773 701	2.272	0.918 665 6	1.354 38	10.113 1	9.907 645	2.284 5	0.920 933 2	0.025	0.930 116	4.271 8	43.202 27	4.351 9	42.503 47	43.581 47	43.202 2
435	1.189 0	9.106 471	2.248	0.914 285 9	1.243 85	9.440 08	9.808 126	2.26	0.916 475 8	0.024	1.016 143	4.465 0	42.150 52	4.334	41.904 68	42.439 35	42.503 4
450	1.081 9	8.453 854	2.224	0.909 880 5	1.135 49	8.780 16	9.710 637	2.236	0.912 083 2	0.024	1.100 505	4.648 7	40.798 87	4.315 3	41.321 09	42.322 22	41.904 6
465	0.977 6	7.818 642	2.201	0.905 634 0	1.029 76	8.136 24	9.615 179	2.212 5	0.907 757 2	0.023	1.182 738	4.817 1	39.193 86	4.297 4	40.752 52	41.198 24	41.321 09
480	0.876 6	7.203 552	2.178	0.901 363 0	0.927 10	7.511 09	9.521 752	2.189 5	0.903 498 5	0.023	1.262 390	4.978 7	37.350 89	4.279 9	40.198 83	41.089 40	40.752 52
495	0.779 3	6.611 216	2.156	0.897 254 5	0.827 10	6.907 38	9.430 357	2.167	0.899 308 8	0.022	1.339 022	5.125 5	35.404 36	4.262 7	39.659 86	39.983 10	40.198 83
510	0.686 2	6.044 167	2.134	0.893 122 8	0.732 79	6.327 69	9.340 992	2.145	0.895 188 7	0.022	1.412 209	5.263 7	33.807 88	4.245 7	39.123 29	39.882 39	39.659 86
525	0.597 6	5.504 831	2.112	0.888 967 7	0.641 95	5.774 49	9.251 627	2.123	0.891 045 2	0.022	1.481 046	5.390 5	31.127 78	4.228 8	38.601 23	39.781 10	39.123 29
540	0.514 0	4.995 516	2.091	0.884 979 1	0.555 85	5.250 17	9.164 294	2.101 5	0.886 973 4	0.021	1.545 642	5.506 8	28.911 97	4.212 1	38.093 55	38.696 04	38.601 23
555	0.453 56	4.518 400	2.07	0.880 968 5	0.474 87	4.756 95	9.078 991	2.080 5	0.882 973 8	0.021	1.605 629	5.612 7	25.699 40	4.195 7	37.600 09	38.602 72	38.093 55
570	0.362 9	4.075 523	2.05	0.877 127 9	0.399 33	4.296 96	8.995 720	2.06	0.879 048 2	0.02	1.660 663	5.708 0	24.527 41	4.179 7	37.120 71	37.534 40	37.600 09
585	0.296 1	3.668 781	2.03	0.873 266 7	0.329 58	3.872 15	8.914 479	2.04	0.875 197 3	0.02	1.710 418	5.792 9	22.431 23	4.164 0	36.655 26	37.448 83	37.120 71
600	0.235 6	3.299 914	2.011	0.869 579 2	0.265 90	3.484 34	8.835 269	2.020 5	0.871 423 0	0.019	1.754 597	5.867 2	20.443 71	4.148 7	36.203 61	36.396 70	36.655 26
615	0.181 5	2.970 500	1.992	0.865 872 4	0.208 57	3.135 20	8.758 091	2.001 5	0.867 725 8	0.019	1.792 927	5.981 0	18.595 04	4.133 7	35.765 61	36.318 65	36.203 61
630	0.134 1	2.681 947	1.974	0.862 342 8	0.157 83	2.826 22	8.682 943	1.983	0.864 107 6	0.018	1.825 163	5.984 1	18.912 48	4.119 0	35.341 13	35.282 15	35.765 61
645	0.093 6	2.435 490	1.956	0.858 795 3	0.113 91	2.558 71	8.609 827	1.985	0.860 569 0	0.018	1.851 088	6.026 4	15.420 04	4.104 7	34.918 32	35.211 38	35.341 13
660	0.060 2	2.232 183	1.938	0.855 229 9	0.076 98	2.333 83	8.536 710	1.947	0.857 012 6	0.018	1.870 015	6.057 2	14.136 52	4.090 3	34.508 85	35.140 25	34.918 92
675	0.034 1	2.072 896	1.921	0.851 845 9	0.047 21	2.152 54	8.465 625	1.929 5	0.853 537 9	0.017	1.882 284	6.077 0	13.081 07	4.076 3	34.508 85	34.122 38	34.508 85
690	0.015 3	1.958 311	1.904	0.848 445 3	0.024 72	2.015 60	8.396 570	1.912 5	0.850 145 8	0.017	1.887 770	6.085 8	12.268 74	4.062 6	34.112 61	34.058 30	94.112 61
705	0.003 9	1.888 918	1.887	0.845 027 9	0.009 62	1.923 61	8.327 516	1.895 5	0.846 736 6	0.017	1.885 875	6.082 8	11.701 03	4.048 9	33.717 86	33.993 91	33.717 86

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Table A.1. Overview of SI Units

	Units	Symbol
Base units		
length	meter	m
mass	kilogram	kg
time	second	s
temperature*	kelvin	K
electrical current	ampere	A
luminous intensity	candela	cd
amount of material	mole	mol
Derived units		
Supplementary units		
angles in the plane	radian	rad
solid angles	steradian	sr
*Use degrees Celsius (°C), which has a more common usage than Kelvin.		

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Table A.2. Relationship of Mass and Weight

	Mass	Weight or Force of Gravity	Force
English	slug pound-mass	pound pound-force	pound pound-force
metric	kilogram	newton	newton

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Table A.3. Derived Units With Special Names

Quantity	Name	Symbol	Expression
Frequency	hertz	Hz	s^{-1}
Force	newton	N	kg × m/s^2
Pressure, stress	pascal	Pa	N/m^2
Energy, work, quantity of heat	joule	J	N × m
Power, radiant flux	watt	W	J/s
Electric charge, quantity	coulomb	C	A × s
Electric potential	volt	V	W/A
Capacitance	farad	F	C/V
Electric resistance	ohm	Ω	V/A
Electric conductance	siemens	S	A/V
Magnetic flux	weber	Wb	V × s
Magnetic flux density	tesla	T	Wb/m^2
Inductance	henry	H	Wb/A
Luminous flux	lumen	lm	cd × sr
Illuminance	lux	lx	lm/m^2

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Table A.4. Useful Conversion Factors

Quantity	From English Units	To Metric Units	Multiplied by*
Length	mile yard foot inch	km m m mm	1.609 0.914 4 0.304 8 25.40
Area	square mile acre acre square yard square foot square inch	km^2 m^2 hectare m^2 m^2 mm^2	2.590 4 047 0.4047 0.836 1 0.092 90 645.2
Volume	acre foot cubic yard cubic foot cubic foot 100 board feet gallon cubic inch	m^3 m^3 m^3 L (1 000 cm^3) m^3 L (1 000 cm^3) cm^3	1 233 0.764 6 0.028 32 28.32 0.236 0 3.785 16.39
Mass	lb kip (1 000 lb)	kg metric ton (1 000 kg)	0.453 6 0.453 6
Mass/unit length	plf	kg/m	1.488
Mass/unit area	psf	kg/m^2	4.882
Quantity	**From English Units**	**To Metric Units**	**Multiplied by***
Mass density	pcf	kg/m^3	16.02
Force	lb kip	N kN	4.448 4.448
Force/unit length	plf klf	N/m kN/m	14.59 14.59
Pressure, stress, modulus of elasticity	psf ksf psi ksi	Pa kPa kPa MPa	47.88 47.88 6.895 6.895
Bending moment, torque, moment of force	ft-lb ft-kip	N × m kN × m	1.356 1.356
Moment of mass	lb × ft	kg × m	0.1383
Moment of inertia	lb × ft^2	kg × m^2	0.042 14
Second moment of area	in^4	mm^4	416 200
Section modulus	in^3	mm^3	16 390
Power	ton (refrig) Btu/s hp (electric) Btu/h	kW kW W W	3.517 1.054 745.7 0.293 1
Quantity	**From English Units**	**To Metric Units**	**Multiplied by***

Volume rate of flow	ft^3/s cfm cfm mgd	m^3/s m^3/s L/s m^3/s	0.028 32 0.000 471 9 0.471 9 0.043 8
Velocity, speed	ft/s	m/s	0.304 8
Acceleration	f/s^2	m/s^2	0.304 8
Momentum	lb × ft/sec	kg × m/s	0.138 3
Angular momentum	lb × ft^2/s	kg × m^2/s	0.042 14
Plane angle	degree	rad mrad	0.017 45 17.45
* 4 significant figures; underline denotes exact conversion			

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Table A.5. Prefixes

Submultiples			Multiples		
deci	10^{-1}	d	deka	10^{1}	da
centi	10^{-2}	c	hecto	10^{2}	h
milli	10^{-3}	m	kilo	10^{3}	k
micro	10^{-6}	μ	mega	10^{6}	M
nano	10^{-9}	n	giga	10^{9}	G
pica	10^{-12}	p	tera	10^{12}	T
femto	10^{-15}	f	peta	10^{15}	P
atto	10^{-18}	a	exa	10^{18}	E
zepto	10^{-21}	z	zetta	10^{21}	Z
yocto	10^{-24}	y	yotto	10^{24}	Y

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Table A.6. Physical Properties of Water at Atmospheric Pressure in SI Units

Temperature		Density	Specific Weight	Dynamic Viscosity	Kinematic Viscosity	Vapor Pressure	Surface Tension[1]	Bulk Modulus
°C Centigrade	°F Fahrenheit	kg/m^3	N/m^3	mPa × s	μ m^2/s	N/m^2 abs.	N/m	GN/m^2
0	32	1 000	9 810	1.79	1.79	611	0.075 6	1.99
5	41	1 000	9 810	1.51	1.51	872	0.074 9	2.05
10	50	1 000	9 810	1.31	1.31	1 230	0.074 2	2.11
15	59	999	9 800	1.14	1.14	1 700	0.073 5	2.16
20	68	998	9 790	1.00	1.00	2 340	0.072 8	2.20
25	77	997	9 781	0.891	0.894	3 170	0.072 0	2.23
30	86	996	9 771	0.797	0.800	4 250	0.071 2	2.25
35	95	994	9 751	0.720	0.724	5 630	0.070 4	2.27
40	104	992	9 732	0.653	0.658	7 380	0.069 6	2.28
50	122	988	9 693	0.547	0.553	12 300	0.067 9	
60	140	983	9 643	0.466	0.474	20 000	0.066 2	
70	158	978	9 594	0.404	0.413	31 200	0.064 4	
80	176	972	9 535	0.354	0.364	47 400	0.062 6	
90	194	965	9 467	0.315	0.326	70 100	0.060 7	
100	212	958	9 398	0.282	0.294	101 300	0.058 9	

[1]Surface tension of water in contact with air.

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Table A.7. Sediment Particles Grade Scale

Size				Approximate Sieve Mesh Openings Per Inch		Class
Millimeters		Microns	Inches	Tyler	U.S. Standard	
4000-2000	-----	-----	160-80	-----	-----	Very large boulders
2000-1000	-----	-----	80-40	-----	-----	Large boulders
1000-500	-----	-----	40-20	-----	-----	Medium boulders
500-250	-----	-----	20-10	-----	-----	Small boulders
250-130	-----	-----	10-5	-----	-----	Large cobbles
130-64	-----	-----	5-2.5	-----	-----	Small cobbles
64-32	-----	-----	2.5-1.3	-----	-----	Very coarse gravel
Size				Approximate Sieve Mesh Openings Per Inch		Class
Millimeters		Microns	Inches	Tyler	U.S. Standard	
32-16	-----	-----	1.3-0.6	-----	-----	Coarse gravel
16-8	-----	-----	0.6-0.3	2 1/2	-----	Medium gravel
8-4	-----	-----	0.3-0.16	5	5	Fine gravel
4-2	-----	-----	0.16-0.08	9	10	Very fine gravel
2-1	2.00-1.00	2 000-1 000	-----	16	18	Very coarse sand
1-1/2	1.00-0.50	1 000-500	-----	32	35	Coarse sand
Size				Approximate Sieve Mesh Openings Per Inch		Class
Millimeters		Microns	Inches	Tyler	U.S. Standard	
1/2-1/4	0.50-0.25	500-250	-----	60	60	Medium sand
1/4-1/8	0.25-0.125	250-125	-----	115	120	Fine sand
1/8-1/16	0.125-0.062	125-62	-----	250	230	Very fine sand
1/16-1/32	0.062-0.031	62-31	-----	-----	-----	Coarse silt
1/32-1/64	0.031-0.016	31-16	-----	-----	-----	Medium silt
1/64-1/128	0.016-0.008	16-8	-----	-----	-----	Fine silt
Size				Approximate Sieve Mesh Openings Per Inch		Class
Millimeters		Microns	Inches	Tyler	U.S. Standard	
1/128-1/256	0.008-0.004	8-4	-----	-----	-----	Very fine silt
1/256-1/512	0.004-0.002 0	4-2	-----	-----	-----	Coarse clay
1/512-1/1024	0.002 0-0.001 0	2-1	-----	-----	-----	Medium clay

1/1024-1/2048	0.001 0-0.000 5	1-0.5	-----	-----	-----	Fine clay
1/2048-1/4096	0.000 5-0.000 2	0.5-0.24	-----	-----	-----	Very fine clay

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Table A.8. Common Equivalent Hydraulic Units

Volume								
Unit	**Equivalent**							
	cubic inch	liter	u.s. gallon	cubic foot	cubic yard	cubic meter	acre-foot	sec-foot-day
liter	61.02	1	0.264 2	0.035 31	0.001 308	0.001	810.6 E - 9	408.7 E - 9
U.S. gallon	231.0	3.785	1	0.133 7	0.004 951	0.003 785	3.068 E - 6	1.547 E - 6
cubic foot	1728	28.32	7.481	1	0.037 04	0.028 32	22.96 E - 6	11.57 E - 6
cubic yard	46 660	764.6	202.0	27	1	0.746 6	619.8 E - 6	312.5 E - 6
meter^3	61 020	1 000	264.2	35.31	1.308	1	810.6 E - 6	408.7 E - 6
acre-foot	75.27 E + 6	1 233 000	325 900	43 560	1 613	1 233	1	0.504 2
sec-foot-day	149.3 E + 6	2 447 000	646 400	86 400	3 200	2 447	1.983	1

Discharge (Flow Rate, Volume/Time)						
Unit	**Equivalent**					
	gallon/min	**liter/sec**	**acre-foot/day**	**foot^3/sec**	**million gal/day**	**meter^3/sec**
gallon/minute	1	0.063 09	0.004 419	0.002 228	0.001 440	63.09 E - 6
liter/second	15.85	1	0.070 05	0.035 31	0.022 82	0.001
acre-foot/day	226.3	14.28	1	0.504 2	0.325 9	0.014 28
feet^3/second	448.8	28.32	1.983	1	0.646 3	0.028 32
meter^3/second	15 850	1 000	70.04	35.31	22.82	1

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Preface : HEC 18

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This is the third edition of HEC 18. It uses metric (SI) units of measurements and <u>contains updated material not included in the other editions dated February 1991 and April 1993 and should be used as the primary reference.</u>

This Federal Highway Administration (FHWA) publication, "Hydraulic Engineering Circular No. 18 (HEC 18), "Evaluating Scour at Bridges," provides procedures for the design, evaluation and inspection of bridges for scour. This document uses metric (SI) units of measurement and is a revision of HEC 18 dated April 1993 which, in turn was an update of the February 1991 edition of HEC 18. The February 1991 edition was an update of the publication titled, "Interim Procedures for Evaluating Scour at Bridges," issued in September 1988 as part of the FHWA Technical Advisory T5140.20, "Scour at Bridges." T5140.20 has since been superseded by T5140.23, "Evaluating Scour at Bridges" October 28, 1991.[6] In addition to using metric (SI) units this circular contains revisions as a result of further scour related developments and the use of the 1993 edition of HEC 18 by the highway community.

The principal changes from the 1993 edition of HEC 18 are:

1. Conversion of the manual into the metric (SI) system.
2. Inclusion of a discussion of backwater effects on contraction scour and the reference line for measuring contraction scour depths (water surface or energy grade line) (Chapter 2 and Chapter 4).
3. Addition of a K_4 factor to the pier scour equation to account for the armoring effect of large particle sizes in the bed material (Chapter 4).
4. The inclusion of an equation to compute the magnitude of the angle of attack coefficient K_2.
5. Additional figures to help clarify the discussions of pier scour of exposed footings; pile caps place at, near, or in the flow; multiple columns skewed to the flow; and scour resulting from pressure flow.
6. Additional discussion in Chapter 4 and Appendix B on computing pressure scour when a bridge deck is submerged.
7. The tidal scour section in Chapter 4 has been expanded to include a method to determine ΔH for constricted waterways developed by Chang et al.[77]
8. The Maryland State Highway Department's method for evaluation of tidal bridges is given in Appendix D.[77]
9. The addition of codes "U" and "T" to Item 113 (scour critical) for bridges with unknown foundations and bridges over tidal waterways, respectively (Appendix E).

10. The information on scour detection equipment has been updated (Appendix F).
11. The addition of an interim procedure for estimating pier scour with debris (Appendix G).
12. Correction of editorial and minor errors in the text and figures.

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<tr><td colspan="3">16. Abstract

This document is the third of HEC 18, i.e., presents the state of knowledge and practice for the design, evaluation, and inspection of bridges for scour. It contains updated material not included in the second edition dated April 1993. This document is a revision to HEC 18 dated April 1993 which, in turn, was an update of HEC 18 dated Feburary 1991 and of the publication, "Interim Procedures for Evaluating Scour at Bridges," issued in September 1988 as part of the FHWA Technical Advisory T 5140.20, "Scour at Bridges." T 5140.20 has since been superseded by T 5410.23, "Evaluating Scour at Bridges" dated October 28,1991. This document contains revisions obtained from further scour-related developments and use of the 1993 edition of HEC 18 by the highway community.

The principal change from the 1993 edition of HEC 18 is the use of metric (SI) units of measurement. Additional changes are: a discussion of backwater effects on contraction of scour and the use of the water surface or the energy grade line as the reference line for measuring contraction scour depths, addition of a coefficient to the pier scour equation to account for the armoring effect of large particle sizes in the bed material, and addition of an equation to compute the coefficient applied to the pier scour equation when there is an angle of attack. Figures have been added to clarify computation of pier scour for exposed footings, pile caps placed in the flow, multiple columns skewed to the flow, and a scour resulting from pressure flow. This document includes a method to compute scour depths for pressure flow when a bridge deck is submerged. A method to compute ΔH for constricted tidal waterways is given along with the procedure used by Maryland SHD to evaluate scour for bridges over tidal waterways. A discussion of computer models to determine the value of the hydraulic variables for scour analysis of bridges over tidal waterways has been added and information on scour detection equipment has been updated based on recent research. Finally, minor errors in the text and figures have been corrected.</td></tr>
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17. Key Words scour design, contraction scour, local scour, scour susceptible, scour critical, clear-water scour, live-bed scour, superflood, bridge inspection, countermeasures, tidal scour.		18. Distribution statement This document is available to the public through the National Technical Information Service, Springfield, VA 22161 (703) 487-4650	
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Symbols

A, C, D, F, G, H, K, L, N, Q, R, S, T, V, W, Y, Z, MISC.

To jump to a specific part of the alphabet, click on the above HotLinks!
Click the Back button to return to the top of this page.
(If the letter you are looking for does not appear in the HotLink list above, then there are no symbol entries for that letter!)

A

a = Pier width, m
A = Maximum amplitude of elevation of the tide or storm surge, m
A_e = Flow area of the approach cross section obstructed by the embankment, m^2
A_c = Cross-sectional area of the waterway at mean tide elevation--half between high and low tide, m^2
= Net cross-sectional area in the inlet at the crossing, at mean water surface elevation, m^2

C

C_d = Coefficient of discharge

D

D = Diameter of the bed material, m
= Diameter of smallest nontransportable particle in the bed material, m
D_m = Effective mean diameter of the bed material in the bridge, mm or m
= $1.25\ D_{50}$
D_{50} = Median diameter of the bed material, diameter which 50% of the sizes are smaller, mm or m
D_{84} = Diameter of the bed material of which 84% are smaller, mm or m
D_{90} = Diameter of the bed material of which 90% are smaller, mm or m
d = Flow depth above lower chord, m

F

Fr = Froude Number $[V/(gy)^{1/2}]$
= Froude Number of approach flow upstream of the abutment
= Froude Number based on the velocity and depth adjacent to and upstream of the abutment
Fr_1 = Froude Number directly upstream of a pier

G

g = Acceleration of gravity (9.81 m/s²)

H

h_{1-2} = Head loss between sections 1 and 2, m

h_c = Average depth of flow in the waterway at mean water elevation, m

H = Height (i.e., height of a dune), m

K

K = Various coefficients in equations as described below

= Conveyance in Manning's equation $\frac{(AR^{2/3})}{n}$, m³/s

= Bottom width of the scour hole as a fraction of scour depth, m

= Coefficient for pier shape

K_o = Velocity head loss coefficient on the ocean side or downstream side of the waterway

K_b = Velocity head loss coefficient on the bay or upstream side of the waterway

K_s = Shields coefficient

K_1 = Correction factor for pier nose shape

= Coefficient for abutment shape

K_2 = Correction factor for angle of attack of flow

= Coefficient for angle of embankment to flow

K_3 = Correction factor for increase in equilibrium pier scour depth for bed condition

K_4 = Correction factor for armoring in pier scour equation

k_1 & k_2 = Exponents determined in Laursen live-bed contraction equation, depends on the mode of bed material transport

k_s = Grain roughness of the bed. Normally taken as the D_{84} of the bed material, m

L

L = Length of pier or abutment, distance between sections, m

L_c = Length of the waterway, m

L' = Length of abutment (embankment) projected normal to flow, m

N

n = Manning's n
n_1 = Manning's n for upstream main channel
n_2 = Manning's n for contracted section

Q

Q = Discharge through the bridge or on the overbank at the bridge, m^3/s
Q_e = Flow obstructed by the abutment and approach embankment, m^3/s
Q_{max} = Maximum discharge in the tidal cycle, m^3/s
= Maximum discharge in the inlet, m^3/s
Q_t = Discharge at any time, t, in the tidal cycle, m^3/s
Q_1 = Flow in the upstream main channel transporting sediment, m^3/s
Q_2 = Flow in the contracted channel, m^3/s. Often this is equal to the total discharge unless the total flood flow is reduced by relief bridges or water overtopping the approach roadway
Q_{100} = Storm-event having a probability of occurrence of one every 100 years, m^3/s
Q_{500} = Storm-event having a probability of occurrence of one every 500 years, m^3/s
q = Discharge per unit width, $m^3/s/m$
= Discharge in conveyance tube, m^3/s

R

R = Hydraulic radius
= Coefficient of resistance

S

SBR = Set-back ratio of each abutment
S_1 = Slope of energy grade line of main channel, m/m
S_f = Slope of the energy grade line, m/m
S_o = Average bed slope, m/m
S_s = Specific gravity of bed material. For most bed material this is equal to 2.65

T

t = Time from the beginning of total cycle, min
T = Total time for one complete tidal cycle, min
= Tidal period between successive high or low tides, s

V

V = Average velocity, m/s
= Characteristic average velocity in the contracted section for estimating a median stone diameter, D_{50}, m/s

V_{max} = Q_{max}/A', or maximum velocity in the inlet, m/s

V_1 = Average velocity at upstream main channel, m/s
= Mean velocity of flow directly upstream of the pier, m/s

V_2 = Average velocity in the contracted section, m/s

V_c = Critical velocity, m/s, above which the bed material of size D, D_{50}, etc. and smaller will be transported

V_{c50} = Critical velocity for D_{50} bed material size, m/s

V_{c90} = Critical velocity for D_{90} bed material size, m/s

V_e = Q_e/A_e, m/s

V_f = Average velocity of flow zone below the top of the footing, m/s

V_i = Approach velocity when particles at a pier begin to move, m/s

V_{max} = Maximum average velocity in the cross section at Q_{max}, m/s

V_R = Velocity ratio

V_* = Shear velocity in the upstream section, m/s
= $(\tau_o/\rho) = (gy_1S_1)^{1/2}$

VOL = Volume of water in the tidal prism between high and low tide levels, m^3

W

W = Bottom width of the bridge less pier widths, or overbank width (set back distance less pier widths, m
= Topwidth of the scour hole from each side of the pier of footing, m

W_1 = Bottom width of the upstream main channel, m

W_2 = Bottom width of the main channel in the contracted section less pier widths, m

ω = Fall velocity of the bed material of a given size, m/s

Y

y = Depth of flow, m. This depth is used in the Neill's and Larson's equation as the upstream channel depth to determine V_c.
= Depth of flow in the contracted bridge opening for estimating a median stone diameter, D_{50}, m

= Amplitude or elevation of the tide above mean water level, m, at time t

y_a = Average depth of flow on the floodplain, m

y_f = Distance from the bed to the top of the footing, m

y_o = Existing depth of flow, m

y_{ps} = Depth of pier scour, m

y_s = Average scour depth, m

y_{sc} = Depth of contraction scour, m

y_1 = Average depth in the upstream main channel or on the floodplain prior to contraction scour, m

= Depth of flow directly upstream of the pier, m

= Depth of flow at the abutment, on the overbank or in the main channel for abutment scour, m

y_2 = Average depth in the contracted section (bridge opening) or on the overbank at the bridge, m

= Average depth under lower cord, m

Z

Z = Vertical offset to datum, m

Misc

τ_2, τ_o = Average bed shear stress at the contracted section, Pa or N/m²

τ_c = Critical bed shear stress at incipient motion, N/m²

γ = Unit weight of water (9800 N/m^3)

ρ = Density of water (1000 Kg/m^3)

ρ_s = Density of sediment (quartz, 2647 kg/m^3)

Θ = Angle of repose of the bed material (ranges from about 30° to 44°)

= Skew angle of abutment (embankment) with respect to flow

= Angle, in degrees, subdividing the tidal cycle

ΔH = Maximum difference in water surface elevation between the bay and ocean side of the inlet or channel, m

Glossary : Hec 18

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abrasion:	Removal of streambank material due to entrained sediment, ice, or debris rubbing against the bank.
afflux:	Backwater, the increase in water surface elevation upstream of a bridge relative to the elevation occurring under natural conditions.
aggradation:	General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.
alluvial channel:	Channel wholly in alluvium; no bedrock is exposed in channel at low flow or likely to be exposed by erosion.
alluvial fan:	A fan-shaped deposit of material at the place where a stream issues from a narrow valley of high slope onto a plain or broad valley of low slope. An alluvial cone is made up of the finer materials suspended in flow while a debris cone is a mixture of all sizes and kinds of materials.
alluvial stream:	A stream which has formed its channel in cohesive or noncohesive materials that have been and can be transported by the stream.
alluvium:	Unconsolidated material deposited by water.
alternating bars:	Elongated deposits found alternately near the right and left banks of a channel.
anabranch:	Individual channel of an anabranched stream.
anabranched stream:	A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars; individual islands or bars are wider than about three times water width; channels are more widely and distinctly separated than in a braided stream.
apron:	Protective material placed on a streambed to resist scour.
apron, launching:	An apron designed to settle and protect the side slopes of a scour hole after settlement.

armor: Surfacing of channel bed, banks, or embankment slope to resist erosion and scour.

armoring: (a) Natural process whereby an erosion-resistant layer of relatively large particles is formed on a streambed due to the removal of finer particles by streamflow; (b) placement of a covering to resist erosion.

articulated concrete mass: Rigid concrete slabs which can move without separating as scour occurs; usually hinged together with corrosion-resistant wire fasteners; primarily placed for lower bank protection.

average velocity: Velocity at a given cross section determined by dividing discharge by cross-sectional area.

avulsion: A sudden change in the channel course that usually occurs when a stream breaks through its banks; usually associated with a flood or a catastrophic event.

backwater: The increase in water surface elevation relative to the elevation occurring under natural channel and floodplain conditions. It is induced by a bridge or other structure that obstructs or constricts the free flow of water in a channel.

backwater area: The low-lying lands adjacent to a stream that may become flooded due to backwater.

bank: The side slopes of a channel between which the flow is normally confined.

bank, left (right): The side of a channel as viewed in a downstream direction.

bank full discharge: Discharge that, on the average, fills a channel to the point of overflowing.

bank protection: Engineering works for the purpose of protecting streambanks from erosion.

bank revetment: Erosion-resistant materials placed directly on a streambank to protect the bank from erosion.

bar: An elongated deposit of alluvium within a channel, not permanently vegetated.

base floodplain: The floodplain associated with the flood with a 100-year recurrence interval.

bed: The bottom of a channel bounded by banks.

bed form:	A recognizable relief feature on the bed of a channel, such as a ripple, dune, plane bed, antidune, or bar. They are a consequence of the interaction between hydraulic forces (boundary shear stress) and the sedimentary bed.
bed layer:	A flow layer, several grain diameters thick (usually two) immediately above the bed.
bed load:	Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer.
bed load discharge	The quantity of bed load passing a cross section of a stream in a unit of (or bed load) time.
bed material:	Material found on the bed of a stream (May be transported as bed load or in suspension).
bedrock:	The solid rock exposed at the surface of the earth or overlain by soils and unconsolidated material.
bed shear (tractive force):	The force per unit area exerted by a fluid flowing past a stationary boundary.
boulder:	A rock fragment whose diameter is greater than 250 mm.
braid:	A subordinate channel of a braided stream.
braided stream:	A stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times water width; braided stream has the aspect of a single large channel within which are subordinate channels.
bridge opening:	The cross-sectional area beneath a bridge that is available for conveyance of water.
bridge waterway:	The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.
bulkhead:	A vertical, or near vertical, wall that supports a bank or an embankment; also may serve to protect against erosion.
caving:	The collapse of a bank caused by undermining due to the action of flowing water. Also, the falling in of the concave side of a bend of which the curvature is changing.

channel:	The bed and banks that confine the surface flow of a stream.
channelization:	Straightening or deepening of a natural channel by artificial cutoffs, grading, flow-control measures, or diversion of flow into a man-made channel.
cellular-block mattress:	Regularly cavitated interconnected concrete blocks with regular cavities placed directly on a streambank or filter to resist erosion. The cavities can permit bank drainage and the growth of vegetation where synthetic filter fabric is not used between the mattress and bank.
channel diversion:	The removal of flows by natural or artificial means from a natural length of channel.
channel pattern:	The aspect of a stream channel in plan view, with particular reference to the degree of sinuosity, braiding, anabranching.
channel process:	Behavior of a channel with respect to shifting, erosion and sedimentation.
check dam:	A low dam or weir across a channel used to control stage or degradation.
choking (of flow):	Excessive constriction of flow which may cause severe backwater effect.
clay:	A particle whose diameter is in the range of 0.00024 to 0.004 mm.
clay plug:	A cutoff meander bend filled with fine grained cohesive sediments.
cobble:	A fragment of rock whose diameter is in the range of 64 to 250 mm.
concrete revetment:	Plain or reinforced concrete slabs placed on the channel bed to protect it from erosion.
confluence:	The junction of two or more streams.
constriction:	A natural or artificial control section, such as a bridge crossing, channel reach or dam, with limited flow capacity in which the upstream water surface elevation is related to discharge.
contact load:	Sediment particles that roll or slide along in almost continuous contact with the streambed.
contraction:	The effect of channel constriction on flow streamlines.

countermeasure:	A measure intended to prevent, delay or reduce the severity of hydraulic problems.
contraction scour:	Scour in a channel or on a floodplain that is not localized at a pier, abutment, or other obstruction to flow. In a channel, contraction scour results from the contraction of streamlines and usually affects all or most of the channel width.
Coriolis force:	The inertial force caused by the Earth's rotation that deflects a moving body to the right in the Northern Hemisphere.
crib:	A frame structure filled with earth or stone ballast, designed to reduce energy and to deflect streamflow away from a bank or embankment.
critical shear stress:	The minimum amount of shear stress required to initiate soil particle motion.
crossing:	The relatively short and shallow reach of a stream between bends; also crossover.
cross section:	A section normal to the trend of a channel.
current:	Water flowing through a channel.
cut bank:	The concave wall of a meandering stream.
cutoff:	(a) A direct channel, either natural or artificial, connecting two points on a stream, thereby shortening the original length of the channel and increasing its slope; (b) A natural or artificial channel which develops across the neck of a meander loop.
cutoff wall:	A wall, usually of sheet piling or concrete, that extends down to scour-resistant material or below the expected scour depth.
daily discharge:	Discharge averaged over one day.
debris:	Floating or submerged material, such as logs or trash, transported by a stream.
deflector:	Alternative term of "spur."
degradation (bed):	A general and progressive lowering of the channel bed due to erosion.
density of water-sediment mixture:	Bulk density (mass per unit volume), including both water and sediment.
depth of scour:	The vertical distance a streambed is lowered by scour below a reference elevation.

dike: An impermeable linear structure for the control or containment of overbank flow. A dike-trending parallel with a streambank differs from a levee in that it extends for a much shorter distance along the bank, and it may be surrounded by water during floods.

dike (groin, spur, jetty): A structure extending from a bank into a channel that is designed to: (a) reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or (b) deflect erosive current away from the streambank (impermeable dike).

dominant discharge: (a) The discharge which is of sufficient magnitude and frequency to have a dominating effect in determining the characteristics and size of the stream course, channel, and bed; (b) That discharge which determines the principal dimensions and characteristics of a natural channel. The dominant formative discharge depends on the maximum and mean discharge, duration of flow, and flood frequency. For hydraulic geometry relationships, it is taken to be the bankfull discharge which has a return period of approximately 1.5 years in many natural channels.

drift: Alternative term for "debris."

eddy current: A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank.

entrenched stream: Stream cut into bedrock or consolidated deposits.

ephemeral stream: A stream or reach of stream that does not flow for parts of the year. As used here, the term includes intermittent streams with flow less than perennial.

erosion: Displacement of soil particles on the land surface due to water or wind action.

erosion control matting: Fibrous matting (e.g., jute, paper, etc.) placed or sprayed on a streambank for the purpose of resisting erosion or providing temporary stabilization until vegetation is established.

estuary: Tidal reach at the mouth of a stream.

fabric mattress: Grout-filled mattress used for streambank protection.

fetch: The area in which waves are generated by wind having a rather constant direction and speed; sometimes used synonymously with fetch length.

fetch length: The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.

fill slope: Side or end slope of an earth fill embankment.

filter: Layer of fabric, sand, gravel, or graded rock placed between bank revetment and soil for the following purposes: (1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; (2) to prevent the revetment from sinking into the soil; and (3) to permit natural seepage from the streambank, thus preventing the buildup of excessive hydrostatic pressure.

filter blanket: A layer of graded sand and gravel laid between fine-grained material and riprap to serve as a filter.

filter cloth: Geosynthetic fabric that serves the same purpose as a granular filter blanket.

fine sediment load (wash load): That part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally, the fine-sediment load is finer than 0.062 mm for sand-bed channel. Silts, clays and sand could be considered wash load in coarse gravel and cobble-bed channels.

flanking: Erosion resulting from streamflow between the bank and the landward end of a countermeasure for stream stabilization.

flashy stream: Stream characterized by rapidly rising and falling stages, as indicated by a sharply peaked hydrograph. Most flashy streams are ephemeral, but some are perennial.

floodplain: A nearly flat, alluvial lowland bordering a stream, that is subject to inundation by floods.

flow-control structure: A structure either within or outside a channel that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.

flow hazard:	Flow characteristics (discharge, stage, velocity, or duration) that are associated with a hydraulic problem or that can reasonably be sonsidered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness of a countermeasure.
flow slide:	Saturated soil materials which behave more like a liquid than a solid. A flow slide on a channel bank can result in a bank failure.
Froude Number:	A dimensionless number that represents the ratio of inertial to gravitational forces. High Froude Numbers can be indicative of high flow velocity and the potential for scour.
gabion:	A basket or compartmentalized rectangular container made of wire mesh, filled with cobbles or other rock of suitable size. Gabions are flexible and permeable blocks with which flow- and erosion-control structures can be built.
geomorphology/morphology:	That science that deals with the form of the Earth, the general configuration of its surface, and the changes that take place due to erosion of the primary elements and in the buildup of erosional debris.
grade-control (sill, check dam):	Structure placed bank to bank across a stream channel (usually with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or headcutting.
graded stream:	A geomorphic term used for streams that have apparently achieved a state of equilibrium between the rate of sediment transport and the rate of sediment supply throughout long reaches. Any change which alters the state of equilibrium can lead to action by the stream to reestablish equilibrium.
groin:	A structure built from the bank of a stream in a direction transverse to the current. Many names are given to this structure, the most common being "spur," "spur dike," "transverse dike," "jetty," etc. Groins may be permeable, semi-permeable, or impermeable.
guide bank:	Preferred term for spur dike.

hardpoint: A streambank protection structure whereby "soft" or erodible materials are removed from a bank and replaced by stone or compacted clay. Some hard points protrude a short distance into the channel to direct erosive currents away from the bank. Hard points also occur naturally along streambanks as passing currents remove erodible materials leaving nonerodible materials exposed.

headcutting: Channel degradation associated with abrupt changes in the bed elevation (headcut) that generally migrates in an upstream direction.

helical flow: Three-dimensional movement of water particles along a spiral path in the general direction of flow. These secondary-type currents are of most significance as flow passes through a bend; their net effect is to remove soil particles from the cut bank and deposit this material on the point bar.

hydraulic radius: The cross-sectional area of a stream divided by its wetted perimeter.

hydraulic problem: An effect of streamflow, tidal flow, or wave action such that the integrity of the highway facility is destroyed, damaged, or endangered.

incised reach: A stretch of stream with an incised channel that only rarely overflows its banks.

incised stream: A stream which has cut its channel through the bed of the valley floor, as opposed to one flowing on a floodplain.

island: A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Islands originate by establishment of vegetation on a bar, by channel avulsion, or at the junction of minor tributary with a larger stream.

jack: A device for flow control and protection of banks against lateral erosion consisting of three mutually perpendicular arms rigidly fixed at the center. Kellner jacks are made of steel struts strung with wire, and concrete jacks are made of reinforced concrete beams.

jack field: Rows of jacks tied together with cables, some rows generally parallel with the banks and some perpendicular thereto or at an angle. Jack fields may be placed outside or within a channel.

jetty: (a) An obstruction built of piles, rock, or other material extending from a bank into a stream, so placed as to induce scouring or bank building, or to protect against erosion; (b) A similar obstruction to influence stream, lake, or tidal currents, or to protect a harbor.

lateral erosion: Erosion in which the removal of material is extended in a lateral direction, as contrasted with degradation and scour in a vertical direction.

launching: Release of undercut material (stone riprap, rubble, slag, etc.) downslope or into a scoured area.

levee: An embankment, generally landward of top bank, that confines flow during high-water periods, thus preventing overflow into lowlands.

littoral drift: The transport of material along a shoreline.

local scour: Scour in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow.

lower bank: That portion of a streambank having an elevation less than the mean water level of the stream.

mattress: A blanket or revetment materials interwoven or otherwise lashed together and placed to cover an area subject to scour.

meander or full meander: A meander in a river consists of two consecutive loops, one flowing clockwise and the other counter-clockwise.

meander belt: The distance between lines drawn tangent to the extreme limits of successive fully developed meanders.

meander length: The distance along a stream between corresponding points at the extreme limits of successive fully developed meanders.

meander loop: An individual loop of a meandering or sinuous stream lying between inflection points with adjoining loops.

meander ratio:	The ratio of meander width to meander length.
meander width:	The amplitude of swing of a fully developed meander measured from midstream to midstream.
meandering channel:	A channel exhibiting a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.
meander scrolls:	Low, concentric ridges and swales on a floodplain, marking the successive positions of former meander loops.
meandering stream:	A stream having a sinuosity greater than some arbitrary value. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops.
median diameter:	The particle diameter of the 50th percentile point on a size distribution curve such that half of the particles (by weight, number, or volume) are larger and half are smaller.
mid-channel bar:	A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.
middle bank:	The portion of a streambank having an elevation approximately the same as that of the mean water level of the stream.
migration:	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
natural levee:	A low ridge formed along streambanks during floods by deposition that slopes gently away from the channel banks.
nominal sediment diameter:	Equivalent spherical diameter of a hypothetical sphere of the same volume as a given stone.
nonalluvial channel:	A channel whose boundary is completely in bedrock.
normal stage:	The water stage prevailing during the greater part of the year.
overbank flow:	Water movement overtop bank either due to stream stage or to inland surface water runoff.

oxbow: The abandoned bow-shaped or horseshoe-shaped reach of a former meander loop that remains after a stream cuts a new, shorter channel across the narrow neck between closely approaching bends of a meander.

perennial stream: A stream or reach of a stream that flows continuously for all or most of the year.

phreatic line: The upper boundary of the seepage water surface landward of a streambank.

pile dike: A type of permeable structure for the protection of banks against caving; consists of a cluster of piles driven into the stream, braced and lashed together.

piping: Removal of soil material through subsurface flow of seepage water that develops channels or "pipes" within the soil bank

point bar: An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop, usually somewhat downstream from the apex of the loop.

poised stream (stable stream): A stream which, as a whole, maintains its slopes, depths, and channel dimensions without any noticeable raising or lowering of its bed. Such condition may be temporary from a geological point of view, but for practical engineering purposes, the stream may be considered stable.

quarry-run stone: Stone as received from a quarry without regard to gradation requirements.

railbank protection: A type of countermeasure composed of rock-filled wire fabric supported by steel rails or posts driven into streambed.

rapid drawdown: Lowering the water against a bank more quickly than the bank can drain without becoming unstable.

reach: A segment of stream length that is arbitrarily bounded for purposes of study.

regime: The condition of a stream or its channel as regards stability. A stream is in regime if its channel has reached a stable form as a result of its flow characteristics.

regime channel: Alluvial channel that has attained, more or less, a state of equilibrium with respect to erosion and deposition.

regime change: A change in channel characteristics resulting from such things as changes in imposed flows, sediment loads or slope.

regime formula: A formula relating stable alluvial channel dimensions or slope to discharge and sediment characteristics.

reinforced-earth bulkhead: A retaining structure consisting of vertical panels and attached to reinforcing elements embedded in compacted backfill for supporting a streambank.

reinforced revetment: A streambank protection method consisting of a continuous stone toe-fill along the base of a bank slope with intermittent fillets of stone placed perpendicular to the toe and extending back into the natural bank.

retard (retarder structure): A permeable or impermeable linear structure in a channel parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank.

revetment: Rigid or flexible armor placed to inhibit scour and lateral erosion. (See bank revetment).

riffle: A natural, shallow flow area extending across a streambed in which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.

riparian: Pertaining to anything connected with or adjacent to the banks of a stream.

riprap: In the restricted sense, layer or facing of broken rock or concrete dumped or placed to protect a structure or embankment from erosion; also the broken rock or concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.

river training: Engineering works with or without the construction of embankment, built along a stream or reach of stream to direct or to lead the flow into a prescribed channel. Also, any structure configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of the stream.

rock-and-wire mattress: A flat or cylindrical wire cage or basket filled with stone or other suitable material and placed as protection against erosion.

roughness coefficient: Numerical measure of the frictional resistance to flow in a channel, as in the Manning's or Chezy's formulas.

rubble: Rough, irregular fragments of materials of random size used to retard erosion. The fragments may consist of broken concrete slabs, masonry, or other suitable refuse.

sack revetment: Sacks (e.g., burlap, paper, or nylon) filled with mortar, concrete, sand, stone or other available material used as protection against erosion.

saltation load: Sediment bounced along the streambed by energy, turbulence of flow, and by other moving particles.

scour: Erosion due to flowing water; usually considered as being localized as opposed to general bed degradation.

scoured depth: Total depth of the water from water surface to a scoured bed level (compare with "depth of scour").

sediment or fluvial sediment: Fragmental material transported, suspended, or deposited by water.

sediment concentration: Weight or volume of sediment relative to quantity of transporting or suspending fluid or fluid-sediment mixture.

sediment discharge: The quantity of sediment that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.

sediment load: Amount of sediment being moved by a stream.

sediment yield:	The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.
seepage:	The slow movement of water through small cracks and pores of the bank material.
seiche:	Long-period oscillation of a lake or similar body of water.
set-up:	Raising of water level due to wind action.
shallow water (for waves):	Water of such a depth that waves are noticeably affected by bottom conditions; customarily, water shallower than half the wavelength.
shoal:	A submerged sand bank. A shoal results from natural deposition on a streambed which has resisted all erosion; thus, the water is of necessity compelled to pass over it.
sill:	(a) A structure built under water, across the deep pools of a stream with the aim of changing the depth of the stream; (b) A low structure built across an effluent stream, diversion channel or outlet to reduce flow or prevent flow until the main stream stage reaches the crest of the structure.
sinuosity:	The ratio between the thalweg length and the valley length of a sinuous stream.
slope (of channel or stream):	Fall per unit length along the channel centerline.
slope protection:	Any measure such as riprap, paving, vegetation, revetment, brush or other material intended to protect a slope from erosion, slipping or caving, or to withstand external hydraulic pressure.
sloughing:	Sliding of overlying material; same ultimate effect as caving, but usually occurs when a bank or an underlying stratum is saturated.
slope-area method:	A method of estimating unmeasured flood discharges in a uniform channel reach using observed high-water levels.
slump:	A sudden slip or collapse of a bank, generally in the vertical direction and confined to a short distance, probably due to the substratum being washed out or having become unable to bear the weight above it.

soil-cement:	A designed mixture of soil and Portland cement compacted at a proper water content to form a blanket or structure that can resist erosion.
sorting:	Progressive reduction of size (or weight) of particles of the load carried down a stream.
spatial concentration:	The dry weight of sediment per unit volume of water-sediment mixture in place or the ratio of dry weight of sediment or total weight of water-sediment mixture in a sample or unit volume of the mixture.
spill-through abutment:	A bridge abutment having a fill slope on the streamward side.
spur:	A permeable or impermeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank.
spur dike/guide bank:	A dike extending upstream from the approach embankment at either or both sides of the bridge opening. Guide banks may also extend downstream from the bridge.
stability:	A condition of a channel when, though it may change slightly at different times of the year as the result of varying conditions of flow and sediment charge, there is no appreciable change from year to year; that is, accretion balances erosion over the years.
stable channel:	A condition that exists when a stream has a bed slope and cross section which allows its channel to transport the water and sediment delivered from the upstream watershed without aggradation, degradation, or bank erosion (a graded stream).
stage:	Water-surface elevation of a stream with respect to a reference elevation.
stone riprap:	Natural cobbles, boulders, or rock dumped or placed as protection against erosion.
stream:	A body of water that may range in size from a large river to a small rill flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.

streambank erosion: Removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to bank erosion.

streambank failure: Sudden collapse of a bank due to an unstable condition such as due to removal of material at the toe of the bank by scour.

streambank protection: Any technique used to prevent erosion or failure of a streambank.

suspended sediment discharge (suspended load): The quantity of sediment passing through a stream cross section in a unit of time suspended by the turbulence of flow.

sub-bed material: Material underlying that portion of the streambed which is subject to direct action of the flow.

submeander: A small meander contained within the banks of a perennial stream channel. These are caused by relatively low discharges after the flood has subsided.

subcritical, supercritical flow: Open channel flow conditions with Froude Number less than and greater than unity, respectively.

tetrahedron: Component of river-training works made of six steel or concrete struts fabricated in the shape of a pyramid.

tetrapod: Bank protection component of precast concrete consisting of four legs joined at a central joint, with each leg making an angle of 109.5° with the other three.

thalweg: The line extending down a channel that follows the lowest elevation of the bed.

tieback: Structure placed between revetment and bank to prevent flanking.

timber or brush mattress: A revetment made of brush, poles, logs, or lumber interwoven or otherwise lashed together. The completed mattress is then placed on the bank of a stream and weighted with ballast.

toe of bank: That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.

toe protection: Loose stones laid or dumped at the toe of an embankment, groin, etc., or masonry or concrete wall built at the junction of the bank and the bed in channels or at extremities of hydraulic structures to counteract erosion.

total sediment load (or total load): The sum of suspended load and bed load or the sum of bed material load and wash load of a stream.

trench-fill revetment: Stone, concrete, or masonry material placed in a trench dug behind and parallel to an eroding streambank. When the erosive action of the stream reaches the trench, the material placed in the trench armors the bank and thus retards further erosion.

turbulence: Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate lines.

uniform flow: Flow of constant cross section and velocity through a reach of channel at a given instant. Both the energy slope and the water slope are equal to the bed slope under conditions of uniform flow.

unit discharge: Discharge per unit width (may be average over a cross section, or local at a point).

unit shear force (shear stress): The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually in units of stress, lb/ft^2.

unsteady flow: Flow of variable cross section and velocity with respect to time.

upper bank: The portion of a streambank having an elevation greater than the average water level of the stream.

velocity: The rate of motion in a fluid on a stream or of the objects or particles transported therein, usually expressed in ft/s.

velocity-weighted sediment concentration: The dry weight of sediment discharged through a cross section during unit time.

wandering channel:	A channel exhibiting a more or less non-systematic process of channel shifting, erosion and deposition, with no definite meanders or braided pattern.
wandering thalweg:	A thalweg whose position in the channel shifts during floods and typically serves as an inset channel that conveys all or most of the stream flow at normal or lower stages.
wash load:	Suspended material of very small size (generally clays and colloids) originating primarily from erosion on the land slopes of the drainage area and present to a negligible degree in the bed itself.
waterway opening width (area):	Width (area) of bridge opening at (below) a specified stage, measured normal to the principal direction of flow.
weephole:	A hole in an impermeable wall or revetment to relieve the neutral stress or pore pressure in the soil.
windrow revetment:	A row of stone placed landward of the top of an eroding streambank. As the windrow is undercut, the stone is launched downslope, thus armoring the bank.
wire mesh:	Wire woven to form a mesh; where used as an integral part of a countermeasure, openings are of suitable size and shape to enclose rock or broken concrete or to function on fence-like spurs and retards.